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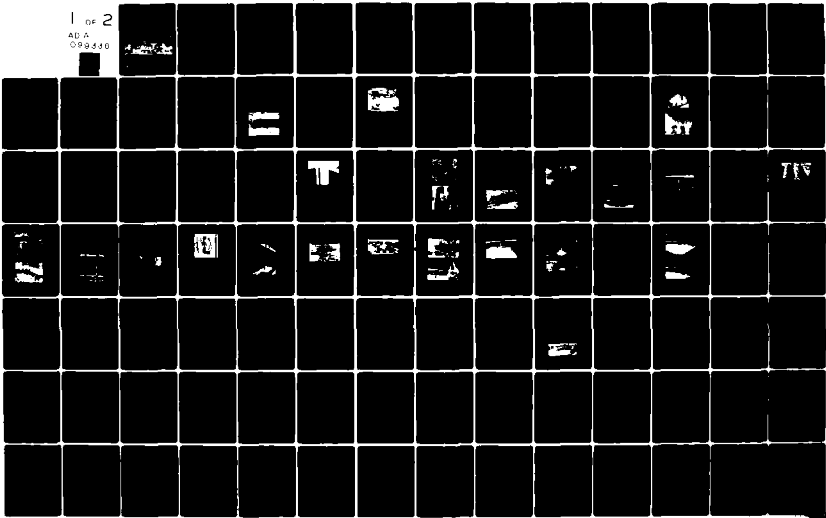

TECHNICAL REPORT EL-81-3

EVALUATION OF UNDERDRAINAGE TECHNIQUES FOR THE DENSIFICATION OF FINE-GRAINED DREDGED MATERIAL

by

David P. Hammer

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Final Report

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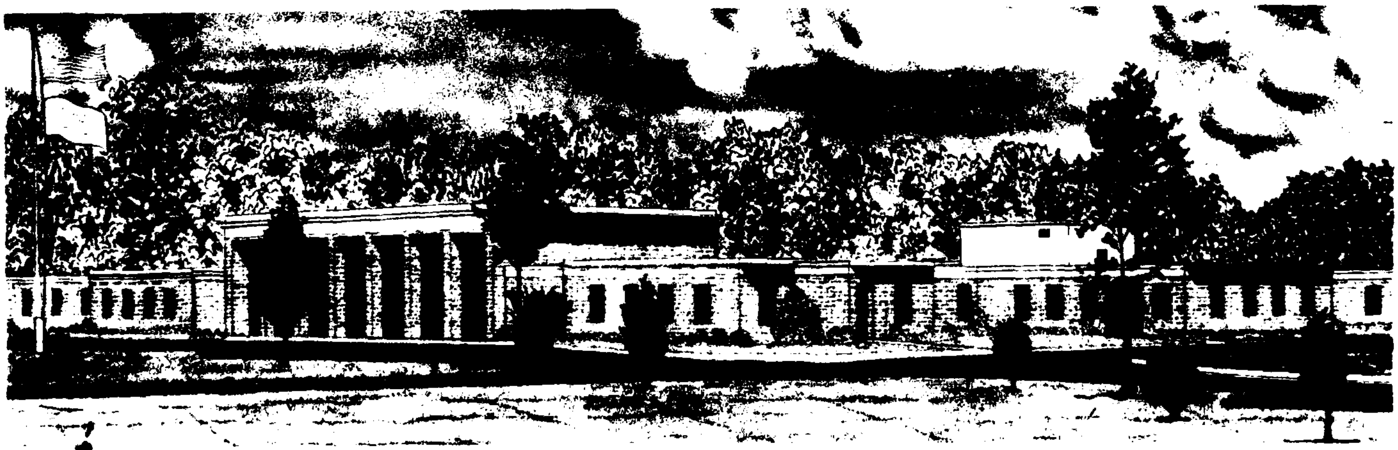

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The result of a large-scale field experiment to evaluate the dewatering/densifying of fine-grained dredged material with underdrainage techniques is herein described. The techniques evaluated were: gravity underdrainage, partial vacuum in an underdrainage layer, seepage consolidation, and seepage consolidation with a partial vacuum in the underdrainage layer.

The experiment was conducted using five test sections having 30- by

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20. ABSTRACT (Continued).

30-ft bottom areas and 1V on 2H sideslopes. One section was used for each of the four techniques evaluated and ~~one~~ section was not treated. The untreated or control section provided a basis against which the effectiveness of the techniques evaluated could be measured. Settlement, pore pressure, water content, and vane shear measurements were taken to provide a basis of evaluation. Initially, a 6-ft nominal thickness lift of dredged material was used; then, after one year, a second 6-ft lift was added. The experiment ran for a total of approximately two years.

All of the techniques evaluated produced more densification than did the test section containing untreated material. Of the techniques evaluated the partial vacuum in an underdrainage layer was the most effective. This was true with both lifts of dredged material tested but was considerably more pronounced with the first lift than the second.

PREFACE

The study reported herein was made by the Geotechnical Laboratory (GL), U. S. Army Engineer Waterways Experiment Station (WES), under the direction of Mr. James P. Sale, Chief, as part of the Corps of Engineers Dredged Material Research Program (DMRP), Disposal Operations Project (DOP), DMRP Work Unit No. 5A15. The DMRP was sponsored by the Office, Chief of Engineers, U. S. Army. The scope of the DMRP work unit was expanded and sponsored by the U. S. Army Engineer District, Chicago. Preparation of the final report was sponsored by the Dredging Operation Technical Support (DOTS) Program. Mr. David P. Hammer prepared this report under the general supervision of Mr. C. L. McAnear, Chief, Soil Mechanics Division, GL.

The DMRP was assigned to the Evironmental Laboratory (EL), under the general supervision of Dr. John Harrison, Chief; the DOP Manager was Mr. Charles C. Calhoun, Jr.; and Dr. T. Allan Haliburton, DMRP Geotechnical Engineering Consultant, was manager for the DOP Work Unit. The DOTS Program Manager is Mr. Calhoun.

The Directors of WES during the work and publication of this report were COL J. L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
horsepower (550 foot-pounds per second)	745.6999	watts
inches	0.0254	metres
mils	0.0000254	metres
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds per square foot	4.882428	kilograms per square metre
pounds per square inch	6894.757	pascals

EVALUATION OF UNDERDRAINAGE TECHNIQUES FOR THE DENSIFICATION OF FINE-GRAINED DREDGED MATERIAL

PART I: INTRODUCTION

Background

1. Dredging to maintain or increase the depths of navigable channels and harbors requires the use of land-based dredged material disposal areas, which are usually formed by encircling the area with dikes. Due to environmental and economic constraints, suitable areas are becoming increasingly difficult to locate. In addition, site conditions may severely limit heights to which retaining dikes can be constructed thus limiting the capacity of the area. Therefore, it is imperative that a disposal area be utilized to the maximum extent possible. One way of accomplishing this is to treat the dredged material to increase and/or accelerate its consolidation (densification) thereby creating additional storage area.

2. This was essentially the overall objective of Research Task No. 5A of the Dredged Material Research Program (DMRP) of the Corps of Engineers (CE), i.e., to test and develop promising techniques for dewatering/densifying dredged material using physical, biological, and/or chemical methods. The study reported herein was initiated as work unit 5A15 under Task 5A and has as its primary objective the field evaluation of various underdrainage techniques. The Dredging Operations Technical Support Program (DOTS) helped fund the completion of this study and analysis of the data.

3. Although the principle reason for densifying dredged material in disposal areas is to increase their storage capacity, secondary benefits can also be accrued that can sometimes be of considerable importance. As the dredged material is densified, its engineering properties are improved, thus making it more suitable as a source of borrow for other projects, or resulting in the disposal area site

becoming attractive as a site for subsequent development.

Purpose

4. This study was conducted to provide an engineering evaluation of dredged material densification by different underdrainage techniques to determine which, if any, of these techniques could be used as a means for increasing storage capacity of disposal areas.

Scope

5. This study is limited to the evaluation of certain dewatering/densification techniques as applied to fine-grained dredged materials having high-water contents after placement in disposal areas. Evaluation of these techniques applied to coarse-grained dredged materials that drain rather rapidly after placement is not included. This report is limited to documentation of the design, construction, operation, and results of the experiment. It does not include analytical analyses of the various techniques evaluated.

Conduct of Experiment

6. This experiment was conducted in six basic phases or steps as outlined in the following tabulation and subsequently described in detail in this report. It was originally planned to evaluate the effectiveness of selected underdrainage techniques on only one lift (6-ft* nominal thickness) of dredged material, but upon completion of this evaluation the U. S. Army Engineer District, Chicago, requested that the techniques be evaluated using two lifts of material. Therefore, a second lift was placed, and the experiment was extended for another year.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

	Phase	Duration
1.	Initial Site Preparation	May-Sept 1976
2.	First Lift Placement	Oct 1976
3.	Data Collection-First Lift	Oct 1976-Oct 1977
4.	Second Lift Site Preparation	Oct 1977
5.	Second Lift Placement	Nov 1977
6.	Data Collection-First and Second Lifts	Dec 1977-Feb 1979

PART II: SELECTION OF DENSIFICATION TECHNIQUES TO BE EVALUATED

7. To initiate this study a comprehensive review was made of Johnson, et al. (1977), a recent state-of-the-art survey of the applicability of conventional densification techniques for soft, fine-grained dredged materials. Of the many techniques discussed, six were chosen as being the most applicable from a practical as well as theoretical standpoint. These methods were then categorically rated (Table 1) as to which exhibited the most promise for prototype application. As a result of this evaluation, as well as overall study of Johnson, et al. (1977), the following methods were selected for field evaluation: (a) underdrainage, (b) seepage consolidation, (c) partial vacuum in underdrainage layer, and (d) combination of (b) and (c), i.e., seepage consolidation with a partial vacuum in the underdrainage layer. Advantages and disadvantages of each method are given in Table 2. The following paragraphs briefly describe each technique selected.

Underdrainage

8. This technique consists simply of providing drainage at the base of the dredged material. Water from the dredged material flows downward into the underdrain by gravity. Stresses for this condition before and after drainage are shown in Figure 1.

Seepage Consolidation

9. In this technique water is ponded on the surface of the dredged material and underdrainage is provided at the base of the dredged material. Downward seepage gradients then act as a consolidating force causing densification (Figure 2).

Partial Vacuum in Underdrainage Layer

10. As in the previous techniques described, drainage is provided at the base of the dredged material, but in addition, a partial vacuum is maintained in the underdrainage layer by pumping from the layer with

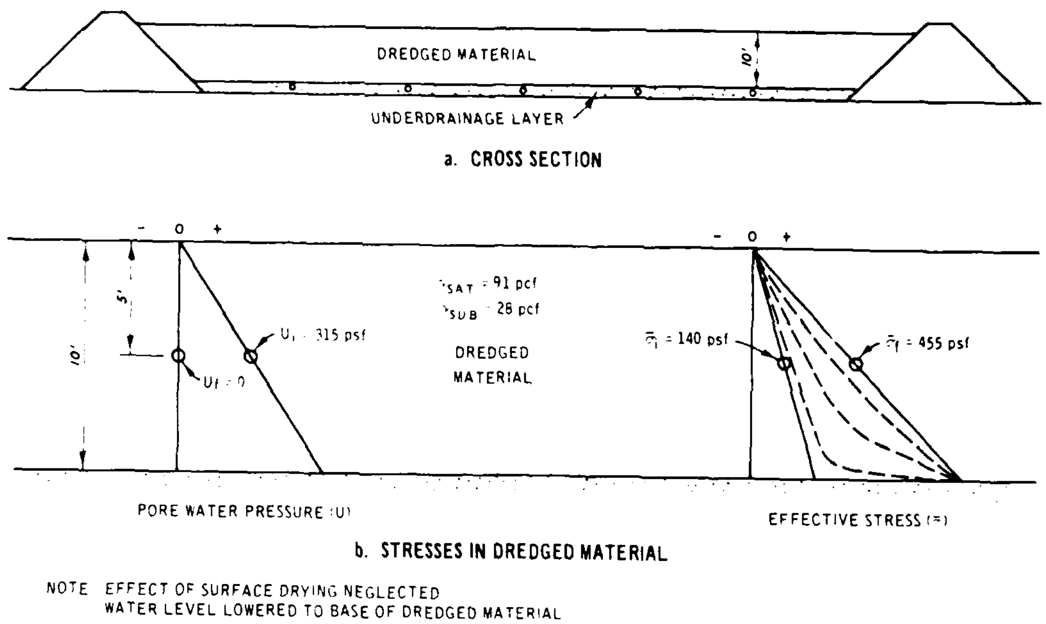


Figure 1. Underdrainage technique

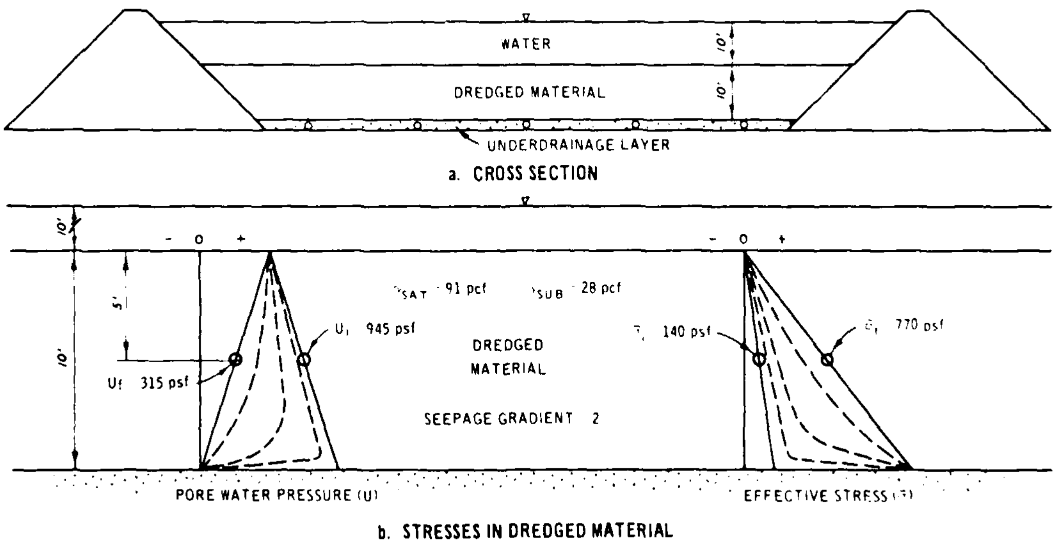


Figure 2. Seepage consolidation technique

vacuum pumps. This results in greatly increased effective stresses in the dredged material as shown in Figure 3.

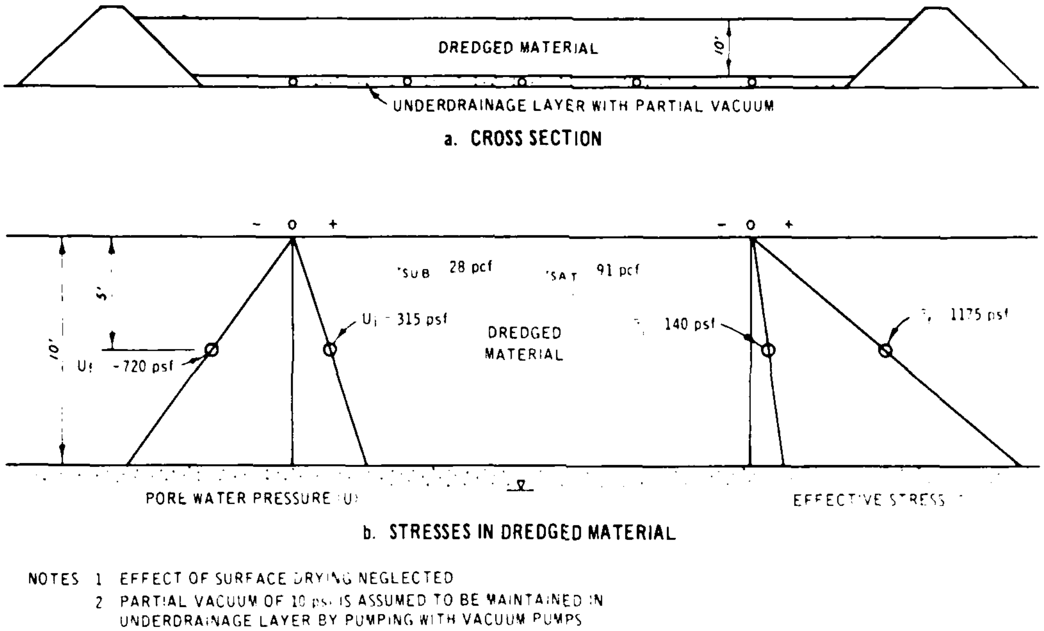


Figure 3. Partial vacuum in underdrainage layer technique

Seepage Consolidation with Vacuum

11. This technique combines the effects of seepage consolidation with those of an induced partial vacuum in an underdrainage layer. Stresses for this condition are shown in Figure 4.

Underdrainage Provisions

12. It should be noted that all of the previously described densification techniques require underdrainage. This is generally accomplished by making use of a naturally occurring pervious foundation or a constructed sand layer. However, these drainage layers must be provided with collector pipes for removal of water. If collector pipes are not used, head losses within the drainage layers would be excessive, thus

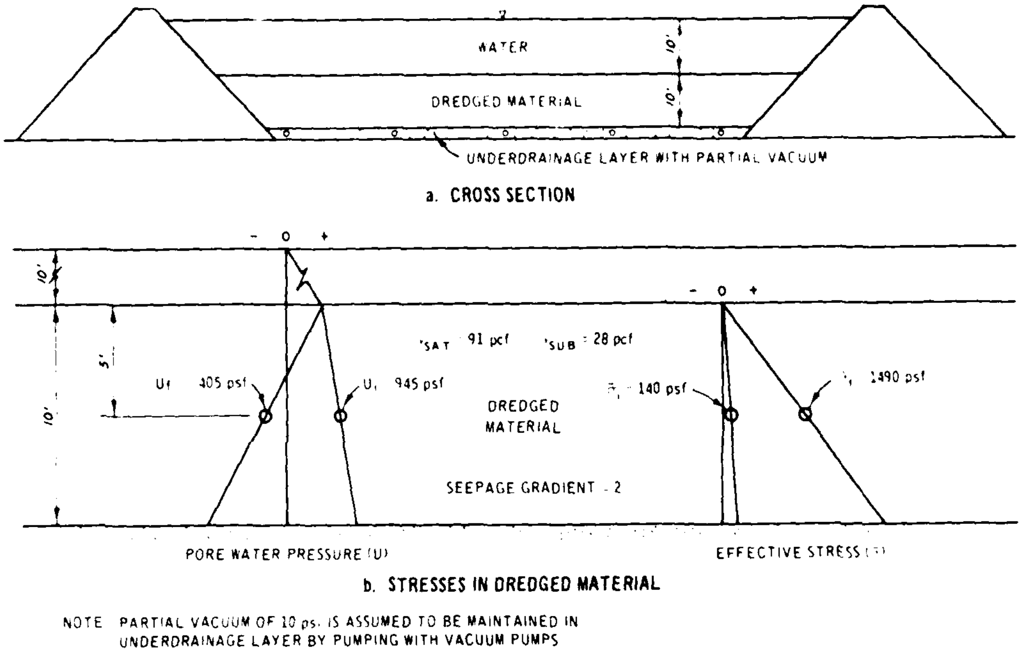


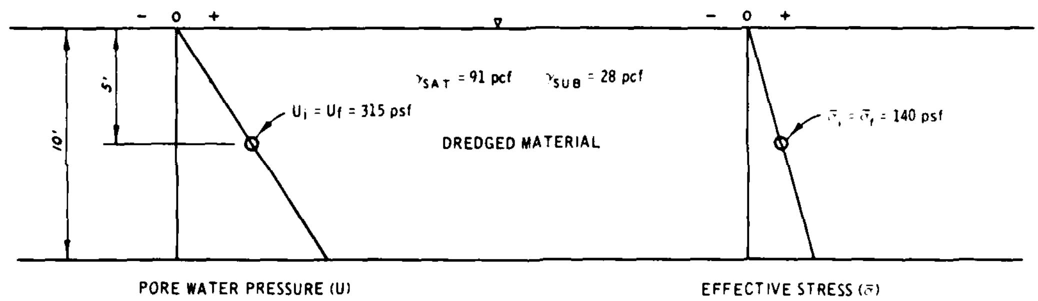
Figure 4. Seepage consolidation with partial vacuum in underdrainage layer technique

prohibiting the drainage layer from functioning as intended. The abovementioned requirements virtually rule out the use of these techniques for dredged material already in existing disposal areas, thus limiting their use to new areas where installation of drainage layers can be accomplished prior to the deposition of dredged material.

No Treatment

13. A disposal area with an impervious foundation functions essentially as a "bathtub" since there is no drainage other than, perhaps, surface drainage. To provide a basis for comparison, stresses for a disposal area receiving no densification treatment are shown in Figure 5. A comparison of the effective stresses shown in Figures 1-5 is given in Table 3.

a. CROSS SECTION



b. STRESSES IN DREDGED MATERIAL

NOTES: 1. EFFECT OF SURFACE DRYING NEGLECTED

2. FINAL STRESSES ARE NOT ULTIMATE STRESSES BUT STRESS CHANGES OCCUR SO SLOWLY FOR THIS CONDITION THAT FOR COMPARATIVE PURPOSES THEY CAN BE CONSIDERED FINAL

Figure 5. No treatment

PART III: EXPERIMENT DESIGN

Site Selection

[illegible] experiment [illegible] area [illegible] U. S. Army [illegible] Upper Pole[illegible] [illegible] (Figure [illegible]) [illegible] dredged material [illegible] the experi[illegible] [illegible] cooperative agreement [illegible]

15. The exact l[illegible] for [illegible] experiment [illegible]

Figure 6. Upper Polecat Bay Disposal Area, Mobile, Ala.

sand in the southeast corner of the disposal area (seen in upper left corner of Figure 6). During past pumpings of dredged material into the area the dredge pipe discharged at this location, and since the heavier sand particles are the first to settle out of the slurry, the sand mound was formed. The sand mound was selected because it would allow excavation and refilling with the dredged material to be tested without presenting stability problems, whereas any other location within the disposal area would be in soft, fine-grained material, which would have required special measures in order for excavation and refilling to take place.

Material Properties

16. The dredged material utilized in this experiment was originally dredged from the Mobile River and deposited in the Upper Polecat Bay Disposal Area, i.e., previously deposited dredged material. The borrow area from which the material was obtained was in the southwest corner of the disposal area (as shown in the upper left corner of Figure 7). A general site plan of the area is shown in Figure 8. The material was excavated by dredging and was pumped to the test site as a slurry through an 8-in.-diam pipeline.

17. Basically, this material was a highly plastic, fine-grained clay (Unified Soil Classification symbol CH), black in color, and containing approximately 6 to 8 percent organic matter.

18. Borings in the borrow area indicated the dredged material to be about 8 to 10 ft in thickness, uniform, and very soft with water contents varying from about 60 to 140 percent. A comprehensive description of material contained in the overall disposal area, which can be considered as representative of material used in this experiment, is found in Palermo (1977).

19. Only index test results are available on the specific material used in the tests since the material was pumped into the test section as a slurry, thereby making it impossible to obtain undisturbed samples. Initial water contents after dredging and placement in the test sections

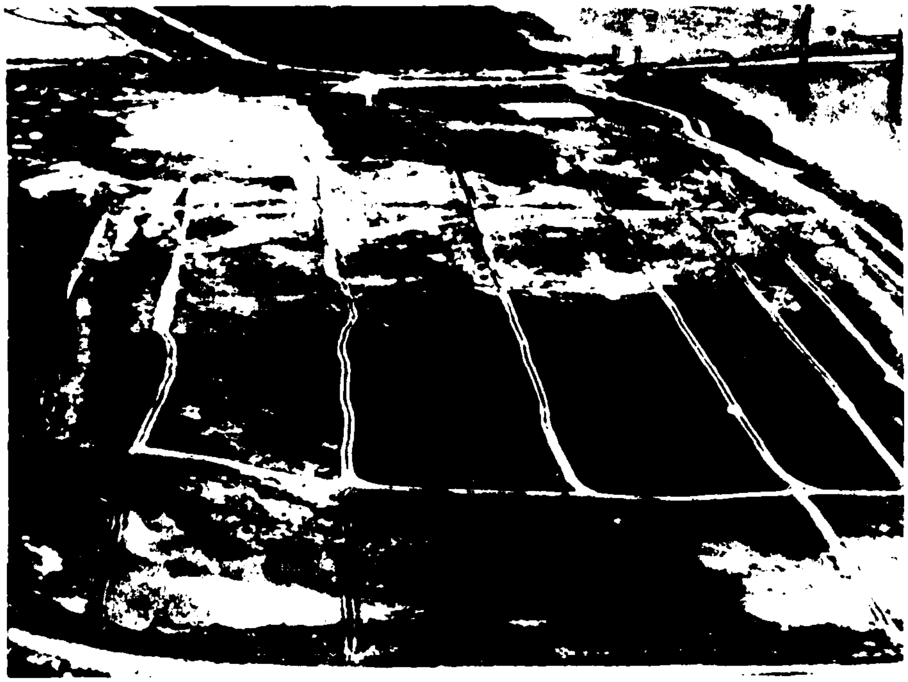


Figure 7. Aerial photograph of test site and borrow area

Figure 8. General site plan

ranged from 400 to 500 percent. These values are believed to be typical for soft, fine-grained materials immediately after deposition by hydraulic means. Results of specific gravity tests for 11 samples ranged from 2.62 to 2.67 and averaged 2.65. Results of Atterberg limits tests are shown in Figure 9 and clearly indicate the highly plastic nature of the material.

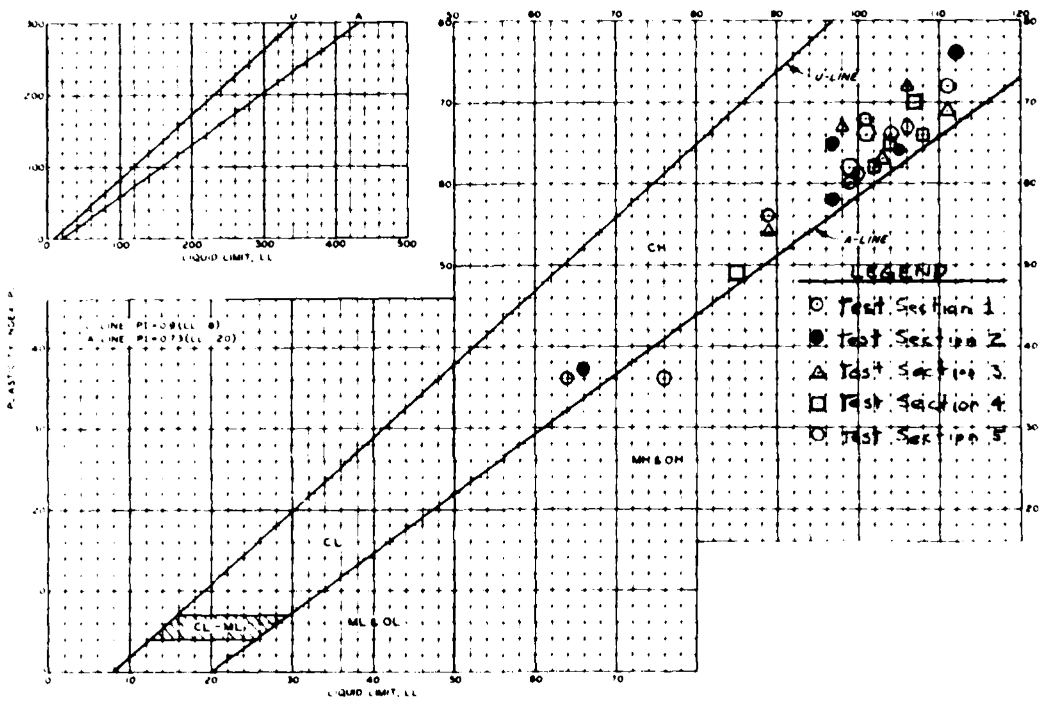


Figure 9. Results of Atterburg limits tests

Test Sections

20. In order to properly evaluate the four methods selected for study, five separate containment areas (subsequently termed test sections) were necessary, one for each technique to be evaluated and one to which no treatment would be applied. The no-treatment section was termed a control section and was necessary to provide a basis of comparison with the four methods being evaluated in order to obtain an accurate measure of their effectiveness.

21. Each test section had to be of sufficient size and the dredged material deposited therein of sufficient depth to avoid scale effects, thus providing as accurate a simulation of prototype behavior as possible. Also, each test section had to contain sufficient instrumentation to properly monitor material behavior. The detailed test section design developed to satisfy these basic requirements is described in the following paragraphs.

Geometry

22. It was decided that a nominal 6-ft depth of dredged material deposited in an excavation having a bottom area of 30 by 30 ft would provide sufficient volume to avoid scale effects. Test section side slopes were designated as 1V on 2H primarily for ease of construction. The lower underdrainage layer used in all test sections except for the control section (which had none) was to be 2 ft thick. A plan and profile of the test sections are shown in Figure 10. Cross sections for all

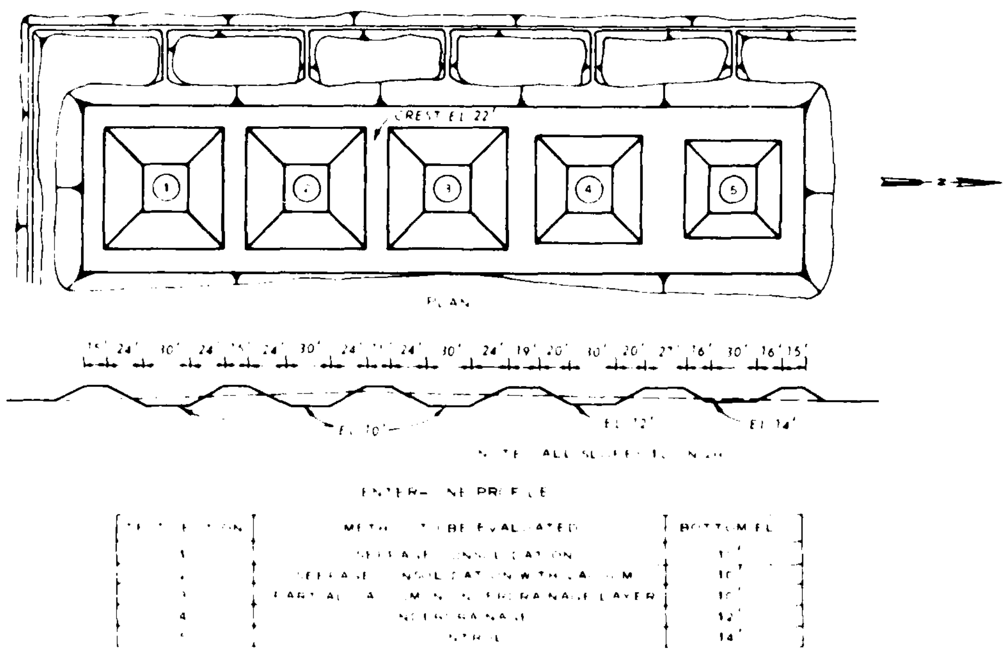


Figure 10. Plan and profile

five test sections are shown in Figure 11.

Impervious liner

23. In order to assure that there would be no flow of water between the sand foundation and material in the test sections, each would be fully lined with two layers of 8-mil-thick polypropylene plastic.

Access bridges

24. To provide access to the test sections, primarily for instrumentation, sampling, and in situ testing, bridges completely spanning each section were necessary. These bridges were designed to consist of 8-in.-diam black iron pipe to which metal framing would be welded which, in turn, would support a plywood deck.

Underdrainage Layer

25. As previously stated, the underdrainage layer in the bottom of each test section (except control) would be 2 ft thick and consist of a pervious medium with a collector pipe system. The underdrainage layer for the seepage consolidation and underdrainage techniques would serve to drain water flowing from the overlying dredged material by gravity. The underdrainage layer for the test sections used to study consolidation with a partial vacuum in the underdrainage layer would also serve as the medium through which the vacuum would be developed. This would be accomplished by installing a closed collector pipe system containing vacuum pumps. Details of the underdrainage layer design are given in the following paragraphs.

Drainage material

26. In order to aid in the determination of which type of drainage material was best suited for this purpose, laboratory filtration tests were performed in 9-in.-diam Lucite cylinders, using dredged material from the Upper Polecat Bay Disposal Area and utilizing five different drainage materials (Figure 12). Since time for performing these tests was minimal, they were not extensive but involved only measurements of the rate at which the water was draining from the dredged material through the drainage material, also, visual observations were made of the

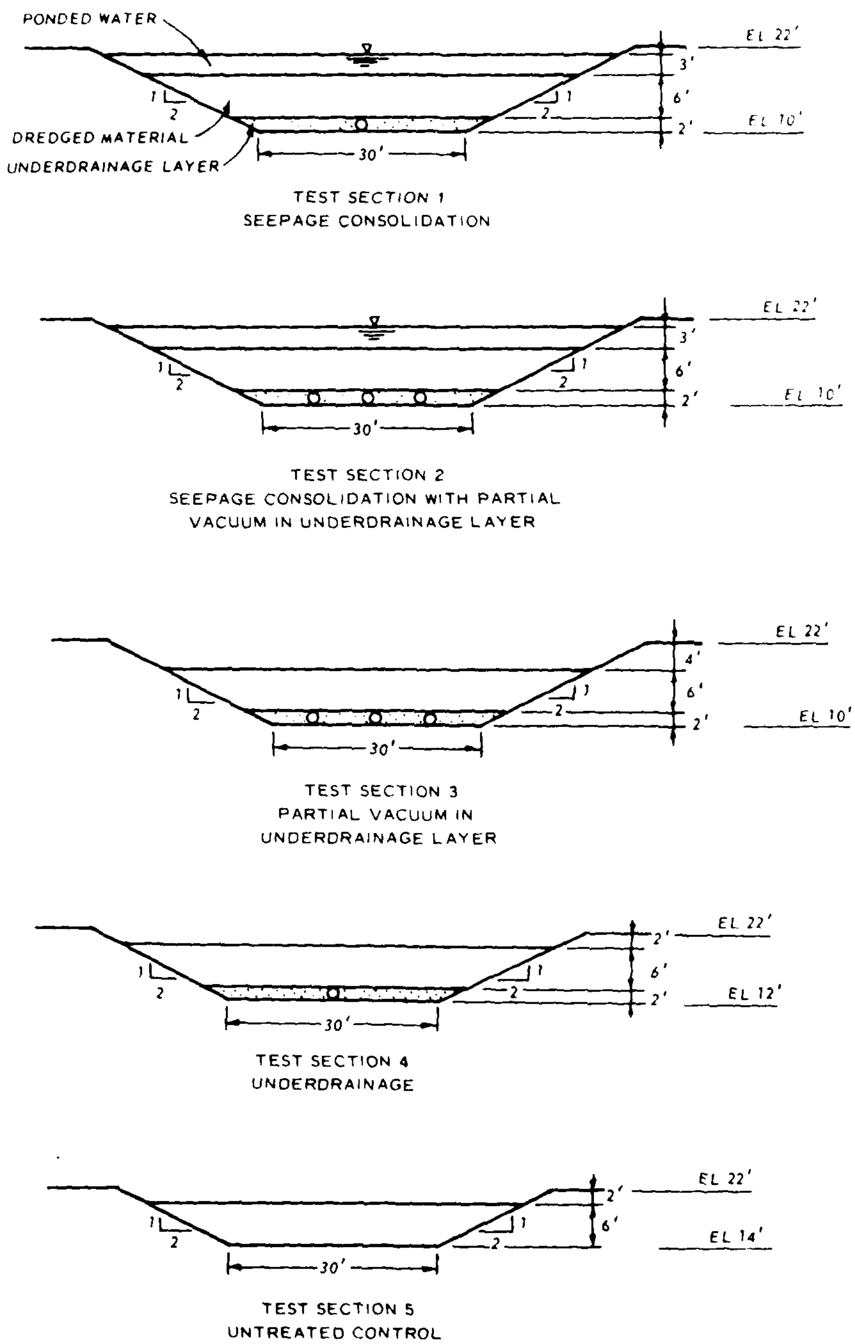
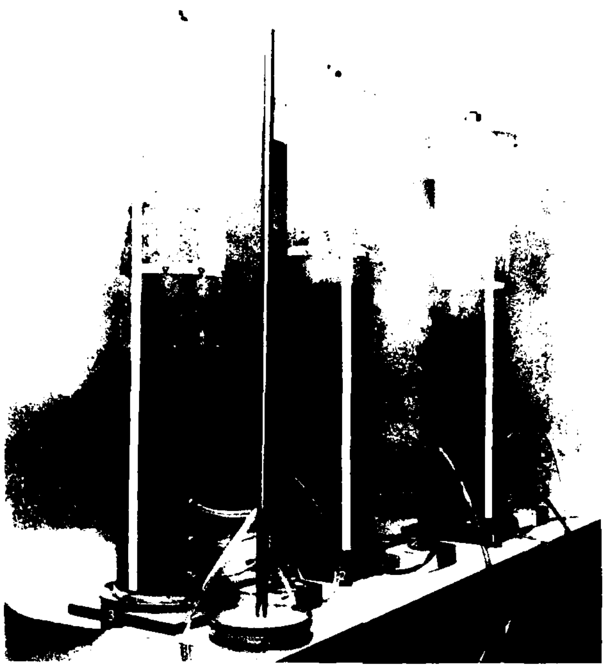
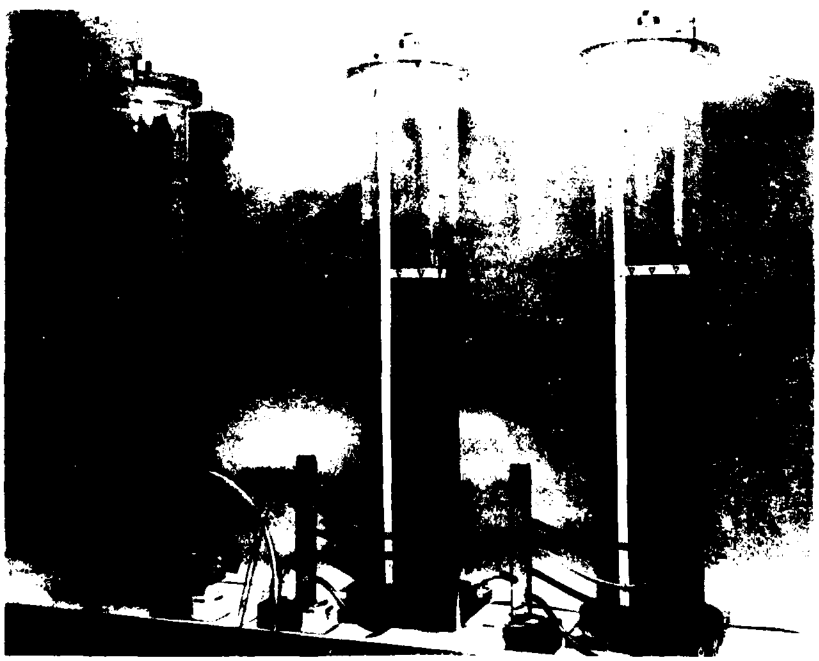


Figure 11. Cross sections; test sections 1-5



a. Tests 1-3



b. Tests 4-6

Figure 12. Laboratory filtration tests

quantity of dredged material which was going into and passing through the drainage media. A summary of these tests is given in Table 4. Drainage rates for all tests were essentially the same because the drainage media did not stay saturated due to the much lower permeability of the overlying dredged material controlling the flow. Based on these data and the observations on penetration of the dredged material into the drainage media, drainage materials in tests 1, 3, and 6 all appeared adequate. Standard concrete sand was selected for use because of availability and the usually lower first cost as compared to pea gravel with filter cloth. Typical gradations for the concrete sand used in these laboratory tests and in the field tests are shown in Figure 13. Even though this sand does not meet Corps of Engineers filter criteria (Headquarters, Department of the Army 1965) when used as a drainage medium for the very fine-grained dredged material, it was considered suitable for use since the only alternative would be to specify a multigraded filter involving more than one drainage material, which would be totally impracticable. More importantly, the laboratory filtration tests showed the concrete sand would work without migration of the dredged material through it, which was the criterion in TM 5-820-2 (Headquarters,

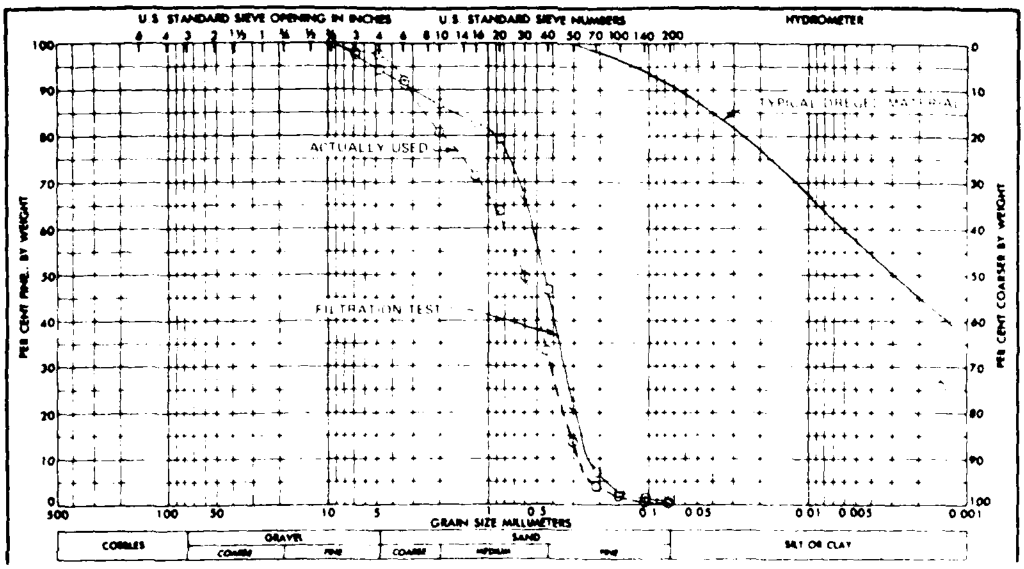


Figure 13. Gradation curves for drainage material and typical dredged material

Department of the Army 1965) that was not met.

Collector pipes

27. All collector pipes were specified to consist of slotted, schedule 40 PVC pipe. Table 5 contains details on the collector pipe system design and Figure 14 shows plan views of the system for test sections 1 and 4 (gravity drainage) and test sections 2 and 3 (vacuum drainage). Collector pipes for the vacuum system were designed by Wellpoint Dewatering Corp., New York, N. Y., as a part of their contract to design and install the entire vacuum system.

28. All collector pipe systems were to be connected to a solid PVC pipe at the inside toe of the test section slope. The solid pipe then would extend under the slope and discharge at the outside toe of the section slope.

Vacuum pumps

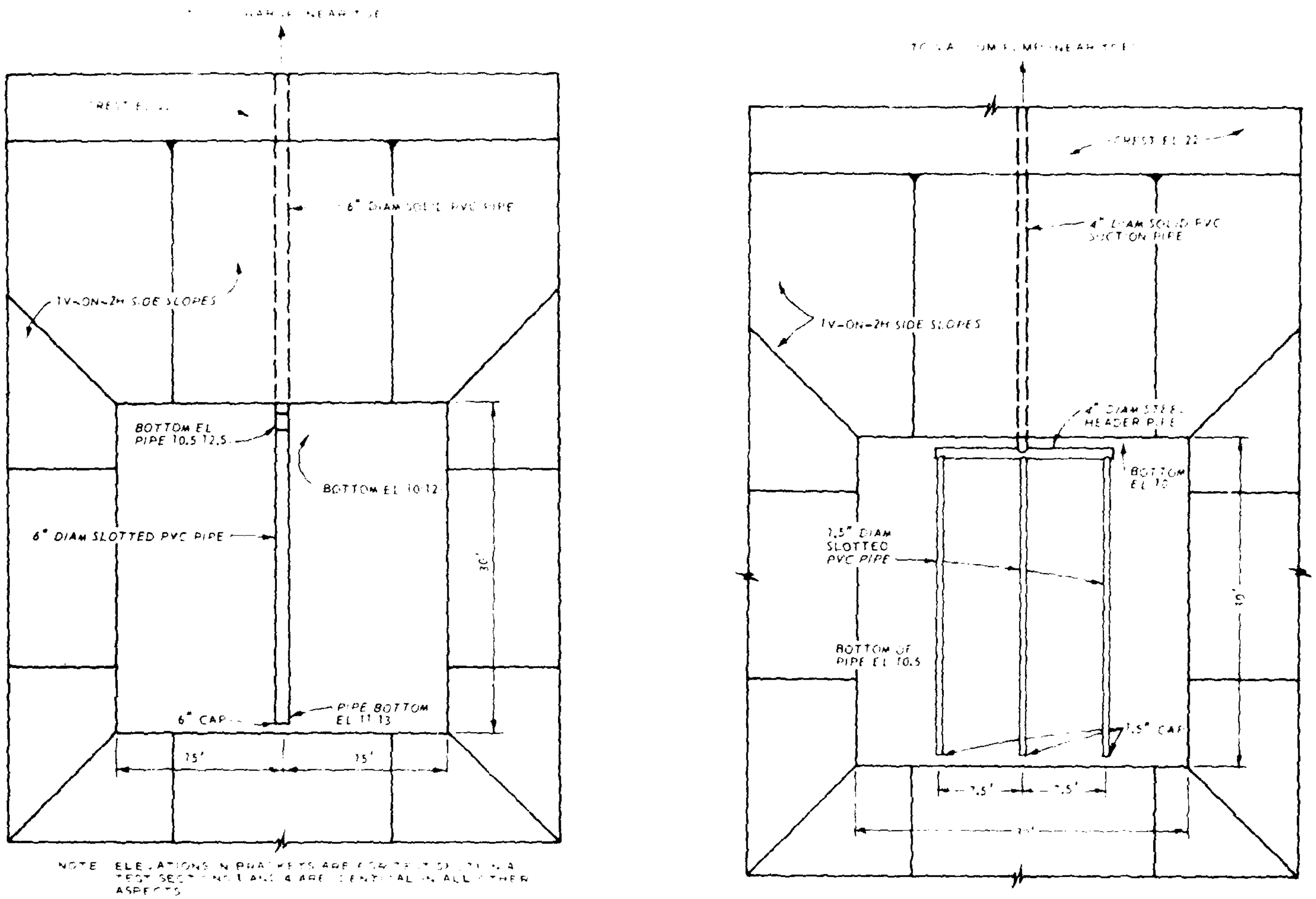
29. Both test sections 2 and 3 were designed to have their own separate vacuum system with provisions made (i.e., piping and valving) to run both sections with either pump if one pump or the other became inoperative. The vacuum pumps themselves were designed to handle approximately 20 cfm of air, each to be driven by a 3-hp electric motor. The vacuum pumps were to be located at the outside slope toe of test sections 2 and 3.

Data Collection and Instrumentation

30. Types of instrumentation to be installed and the measurements each was intended to make are summarized in Table 6. The following paragraphs contain a brief discussion of each type of instrument.

Pore water pressure

31. Three types of piezometers were selected in order to provide two complete systems, each capable of measuring either negative or positive pore water pressure. The porous stone piezometers to measure positive pore water pressure and the tensiometers to measure negative pore water pressure together made one complete system, while the U. S. Army Engineer Waterways Experiment Station (WES) transducer piezometer, which



a. Test sections 1 and 4 (gravity)

b. Test sections 2 and 3 (vacuum)

Figure 14. Plan views of collector pipe systems

is capable of measuring both positive and negative pore water pressure, comprised the second system. All these piezometers were designed to be hung on instrument stands in the middle of each test section at the nominal levels shown in Figure 15.

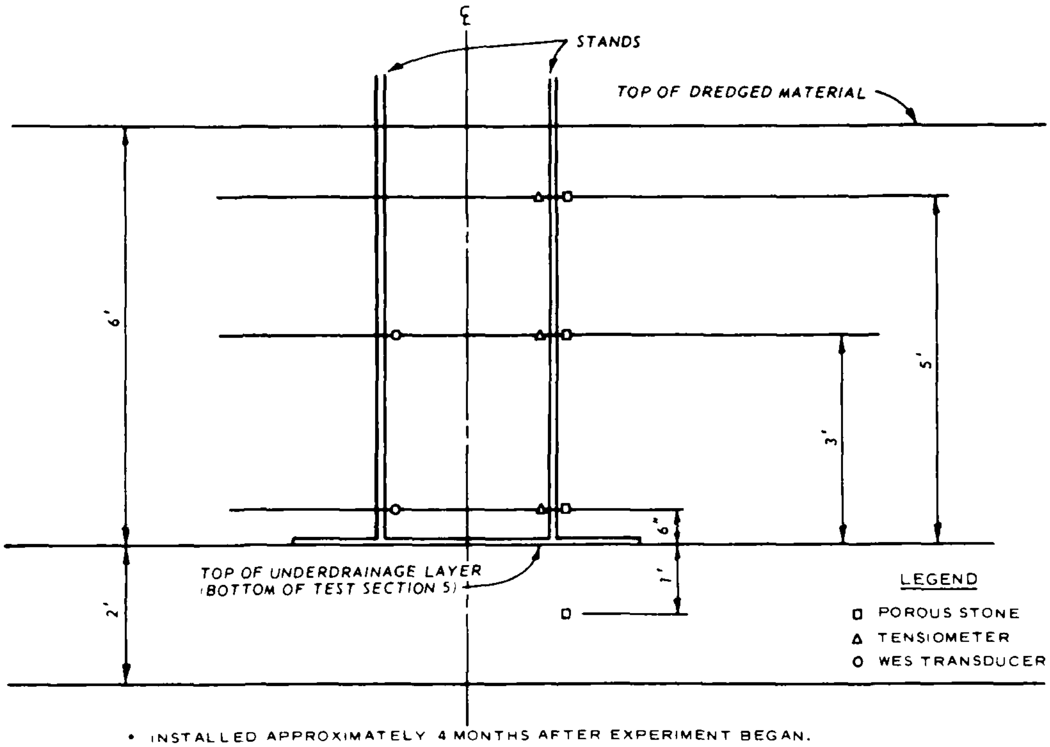


Figure 15. Nominal piezometer locations

32. Porous stone piezometers. The porous stone piezometers consisted of a porous stone approximately 6 in. long, plugged with a rubber stopper in the bottom, and connected to a 1/2-in.-diam PVC riser pipe at the top. The entire stone was to be surrounded by sand contained in a filter cloth bag to prevent migration of the dredged material particles into the stone and to allow backflushing of the instrument if needed. This type of piezometer is described in detail in EM 1110-2-1908 (Department of the Army, Office, Chief of Engineers 1971b).

33. Tensiometers. These instruments consist simply of a porous ceramic cup on the end of a 1/2-in.-diam plastic riser tube. A gage capable of reading soil tension is installed on the upper end of the

riser tube. These types of instruments are commonly used in agriculture to measure soil suction.

34. WES transducers. WES transducer-type piezometers are also described in EM 1110-2-1908. These instruments measure pore water pressure by means of a pressure transducer mounted in a waterproof brass housing. A high-air-entry ceramic porous stone protects the transducer diaphragm. Instrument readout is through an electrical cable attached to an ohmmeter.

Vacuum

35. Instruments to monitor vacuum in the underdrainage layer of test sections 2 and 3 consist of a 6-in.-long, 1.5-in.-diam piece of slotted PVC pipe (same as that used for collector pipe), capped on one end and connected to 1/4-in.-diam plastic tubing on the other end (Figure 16). The tubing was to be brought out to an accessible point where it would be connected to a vacuum gage readout. Twelve of these instruments were to be installed at middepth of the underdrainage layer in both test sections 2 and 3 at the locations shown in Figure 17.

Settlement of underdrainage layer and foundation

36. Settlement plates were to be used to monitor settlement of the underdrainage layer and foundation. These plates consist of 4-ft-square metal meshing attached to an angle-iron frame. A riser pipe consisting of 4-in.-diam perforated PVC pipe would be attached to the plate to provide access to the surface. A crossbar would be inserted through the top of the PVC pipe to provide a point upon which level readings could be taken. These instruments would also serve as stands upon which the WES transducer piezometer could be hung.

Underdrainage discharge

37. Test sections 1 and 4 (gravity). The discharge from test sections 1 and 4 was to be deposited in a covered sump containing an electric sump pump with automatic on-off floats. An hour-meter would be attached in the electrical line so that whenever the sump pump was operating its time would be recorded. The sump pump had been previously calibrated so that its pumping rate was kn . Thus, knowing the pumping

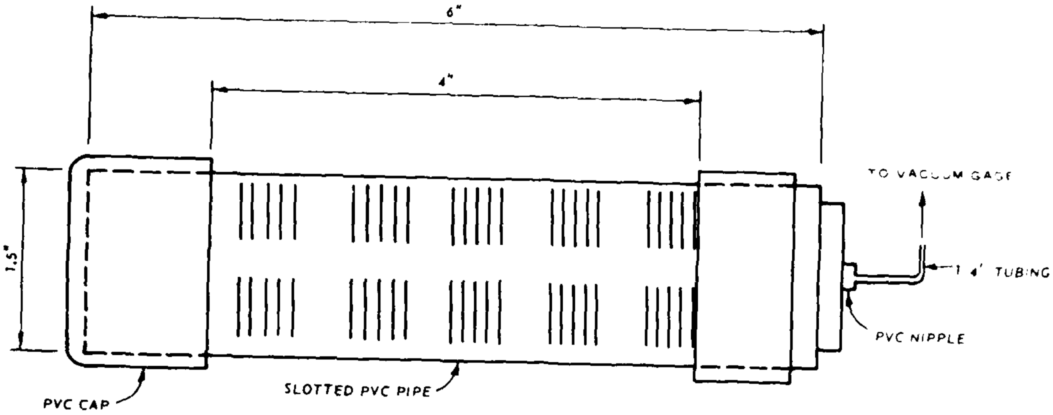


Figure 16. Vacuum monitoring device

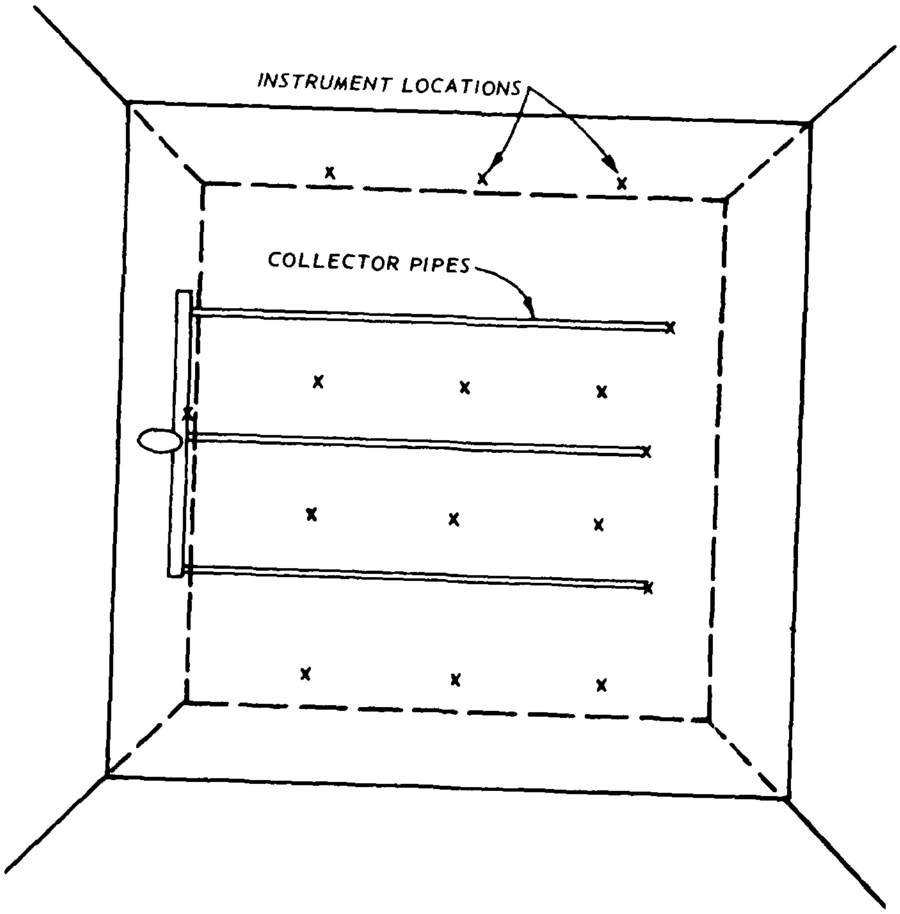


Figure 17. Location of vacuum monitoring devices in underdrainage layer of test sections 2 and 3

rate and the length of operating time, the quantity of discharged water could be calculated.

38. Test sections 2 and 3 (vacuum). Flow meters similar to those employed to measure household water usage were to be attached to the discharge line of the vacuum pumps in order to monitor their discharge.

In situ measurements

39. In addition to the permanently installed instruments previously described, in situ measurements to determine soil properties were also planned. The following paragraphs contain a brief descrition of each measurement.

40. Water content and density. In order to obtain in situ values of dredged material water content and density, the decision was made to utilize a downhole nuclear moisture-density meter, though there was recognition that it might not be effective due to the anticipated wide range of water content and density, and the organic matter contained in the dredged material.

41. To properly utilize this instrument the installation of a 2-in. steel casing into which the instrument would be lowered for readings would be necessary. The casing was to be held upright by being bolted to a 3-ft-square metal grate. Two casings per test section would be installed.

42. Vane shear. Due to the softness of the dredged material which would make undisturbed sampling virtually impossible in the early stages of the experiment and very difficult even in the later stages, a 6-in.-high by 4-in.-diam filed vane (Figure 18) was to be used for determination of in situ shear strength of the dredged material. The vane was to be utilized with 3/4-in.-diam rod and rotated by means of a torque wrench.

43. Surface settlement. Measurements were to be taken to determine the elevation of the dredged material surface in each test section in order to monitor the consolidation of the dredged material in each test section with time. This was to be accomplished by taking level readings with a level instrument and rod at several different locations on the surface of the dredged material in each test section.

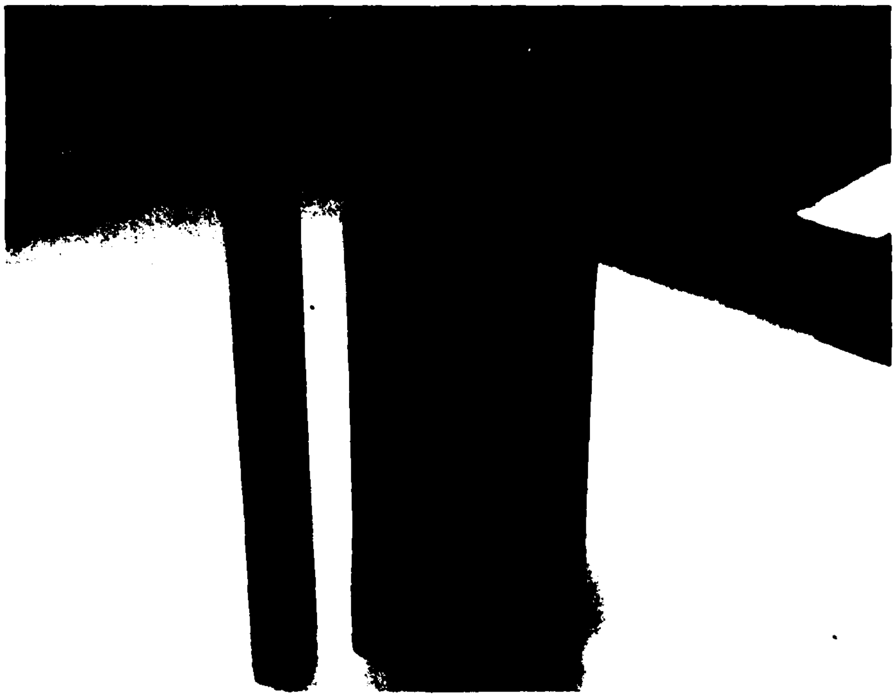


Figure 18. Vane shear device

Sampling and Laboratory Testing

Disturbed samples

44. Samples of the dredged material in each test section were to be taken on a regular basis with every 1 ft of depth. These samples would then be subjected to laboratory water content tests. This procedure would result in a record of the change in water content of the dredged material with time. In addition, disturbed-type sampling would also be accomplished for the purpose of performing Atterberg limits and specific gravity testing.

45. Two types of samplers were to be used to obtain disturbed samples, the slurry sampler developed during the DMRP and the Hvorslev sampler. The slurry sampler would be used to sample very high-water content material (i.e., material still essentially in a slurry state) while the Hvorslev sampler would be used for lower water content or firmer material.

46. Slurry sampler. The lower part of the slurry sampler is shown in Figure 19 and consists of a piece of 2-1/2-in. aluminum conduit containing a 1/4-in. plunger rod on the end of which was attached a rubber stopper. To operate, the device is lowered to the elevation at which the sample is desired. With the stopper in place against the opening in the lower end of the outer pipe, the inside plunger rod is pushed to force the stopper out. Slurry then flows into the outer pipe. The inside pipe (plunger) is then pulled up forcing the stopper into the outer pipe and trapping the sampled material inside. The whole device is then pulled and the sample allowed to flow out into a container.

47. Hvorslev sampler. The Hvorslev sampler is a hand-operated fixed-piston vacuum sampler which obtains a 1-7/8-in.-diam sample (Figure 20). This sampler and its operating instructions are described in EM 1110-2-1907 (Department of the Army, Office, Chief of Engineers 1972).

Undisturbed samples

48. Undisturbed samples of the dredged material would be taken after enough consolidation had occurred to enable such sampling to be accomplished. This type of sampling would also be accomplished with the previously described Hvorslev sampler for the purpose of performing laboratory unconfined compression and Q-triaxial tests. Also, plans were made to use the same sampler, but fitted with a larger diameter sample tube, to obtain samples for laboratory consolidation tests.

Figure 19. Slurry sampler

Figure 20. Hvorslev sampler and sample

PART IV: TEST SITE CONSTRUCTION

49. Part IV presents a description of the design deemed necessary to meet the objective of the experiment and contains a description of actual equipment construction as well as photographs of many of the previously described items. In some cases deviations from the initial design were necessary as construction progressed. Where such deviations occur they are so noted and the reasons for their occurrence given.

Site Preparation

Ditching

50. Since the sand mound where the test sections were to be built contained a perched water table (due to thin layers of fine-grained material caused by dredging shutdowns), it was necessary to drain the area prior to excavation. This was accomplished by a series of ditches excavated with a dragline.

Excavation

51. A dragline was also used for rough excavation and filling operations immediately after the ditching was complete. The site following completion of ditching and rough grading is shown in Figure 21.

Figure 21. Aerial photo of test area following completion of ditching and rough grading

52. Fine grading was accomplished with a dragline and small dozers. Each test section was built to grade except for the back slope where the original drainage ditch was located (Figure 22). This slope

Figure 22. Typical test section before closure of back slope

was left open for installation of the solid portion of the collector pipe system which would carry water from the underdrainage layer to an exit point. Figure 23 shows a test section after installation of the pipe and closure of the section.

Impervious Liner and Access Bridge

Impervious liner

53. Following hand raking of all slopes, two layers of clear [illegible] thick polypropylene plastic were placed in each test section to [illegible] imperviousness (Figure 24). This liner was continuous except where the drainage pipe came through to connect with the slotted collector pipe. At this point a waterproofing compound was placed around the pipe and a concrete collar poured in order to ensure a waterproof seal. Sandbags were used to secure the liner at the crest of each test section. [illegible]

[illegible] test section following installation of [illegible]

[illegible]

liner was proof-tested by filling the test sections with water and monitoring water levels for approximately a week before draining.

54. After the impervious liner was in place, access bridges fabricated on site were moved by a crane into place across the test section (Figure 25).

[illegible] Access bridges, in place across test sections

Underdrainage [illegible]

[illegible]

[illegible] The slotted collector pipes, [illegible] allowed the test section to drain by gravity, were installed [illegible] of the sand drain [illegible] [illegible] blowed [illegible] classed [illegible] [illegible] slotted [illegible]

Figure 26. Placement of drainage material in test section 1

Test sections 2 and 3

56. The sand underdrainage material was placed first for these two test sections (vacuum). Trenches were then dug in the sand, the slotted collector pipes placed (Figure 27), the trenches backfilled, and the sand smoothed to final elevation. Figure 28 shows a typical test section with completed underdrainage layer. Table 7 summarizes final surface elevations of the underdrainage layers for test sections 1-4.

Vacuum and sump pumps

57. Following completion of the underdrainage layer with collector system, sumps and sump pumps were installed in test sections 1 and 4 and vacuum pumps in test sections 2 and 3. All of these pumps were located at the outside toe of their respective test section slopes. Figures 29 and 30 show typical completed systems of sumps and sump pumps and vacuum pumps, respectively.

Figure 27. Slotted collector pipe in trenches (test section 2)

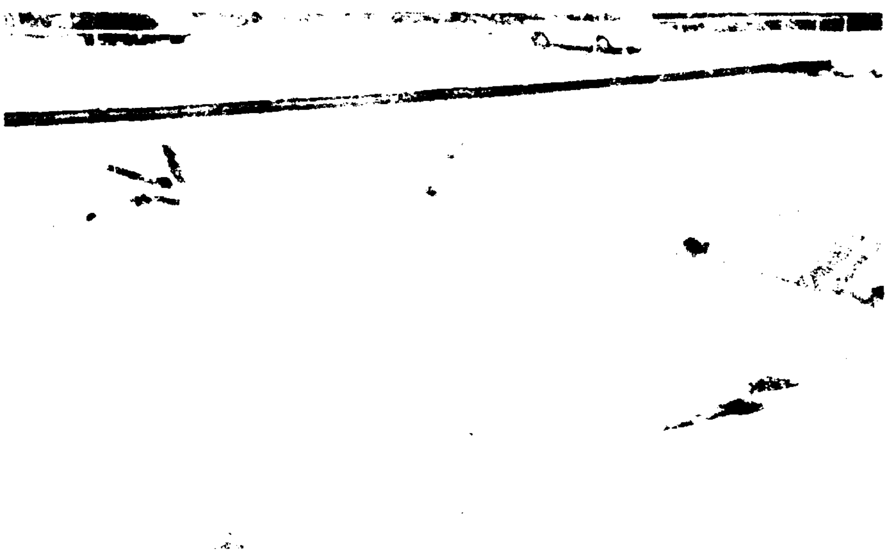


Figure 28. Completed underdrains [illegible]

Figure 29. Sump containing sump pump used to pump out drainage layer discharge from test sections 1 and 4

Figure 30. Typical vacuum pump installation

Instrumentation

[illegible]

[illegible]

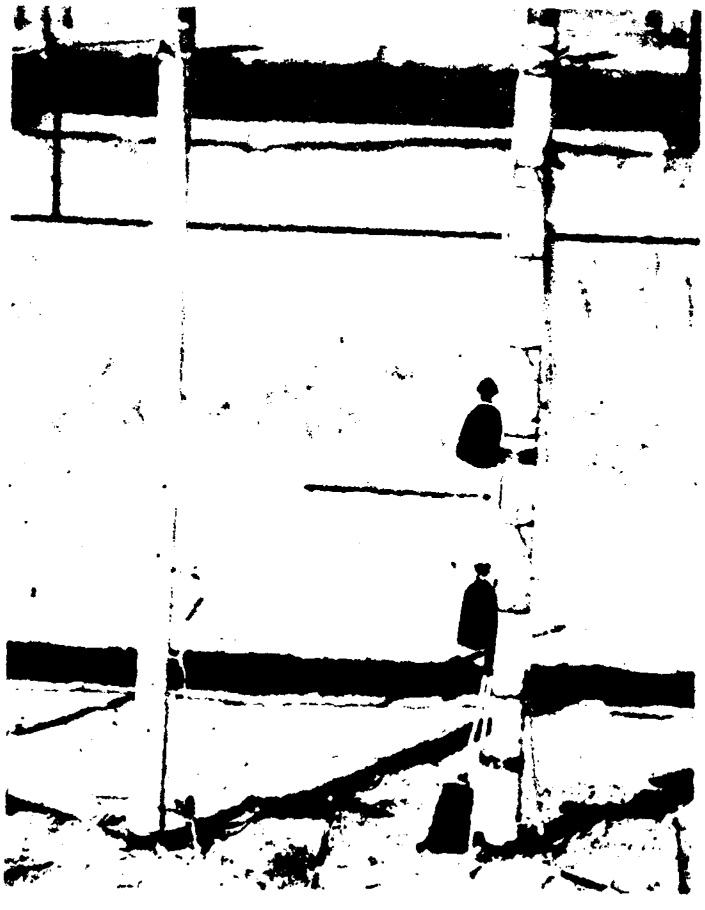


Figure 31. Piezometers in place on a settlement plate riser and on an instrument stand

ceramic stones were de-aired by boiling and then installed in the unit underwater where they remained until a prophylactic membrane filled with de-aired water was placed over the entire unit to ensure that the stone remained saturated. The units were then placed on the instrument stand just prior to filling (Figure 32), and the prophylactic membrane was removed only after submergence of the instrument was attained during

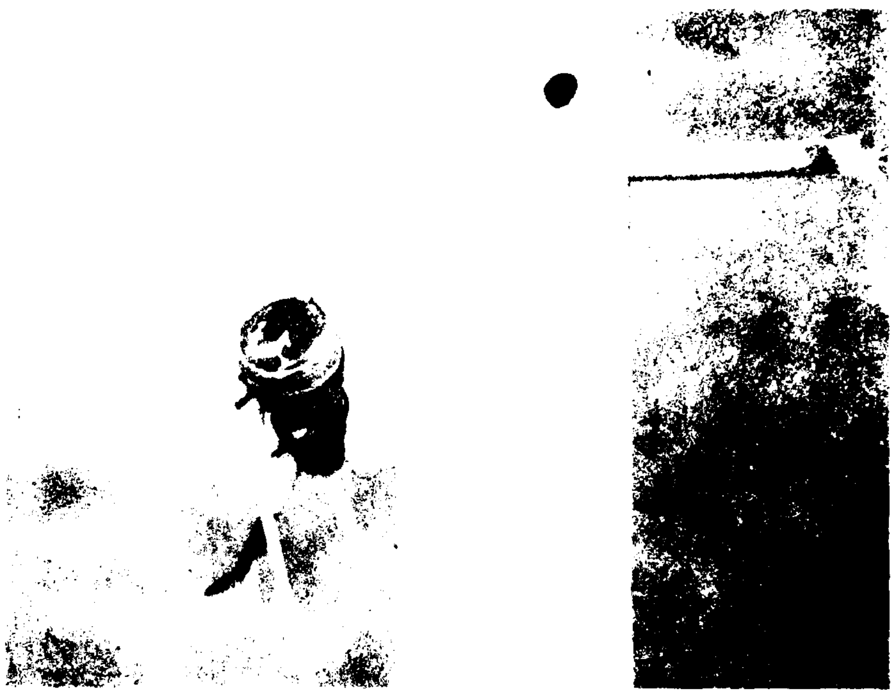


Figure 32. In place WES transducer piezometer

filling. The electrical cables were strung along the access bridge to a readout box on the crest, as shown in Figure 33. The instruments were calibrated and a zero reading was obtained just prior to filling. The leads were protected by waterproof fittings.

60. Vacuum. The instruments to measure vacuum in the underdrainage layer were installed just after final placement of the sand drainage material. Tubing from each instrument was carried to the instrument readout box shown in Figure 33.

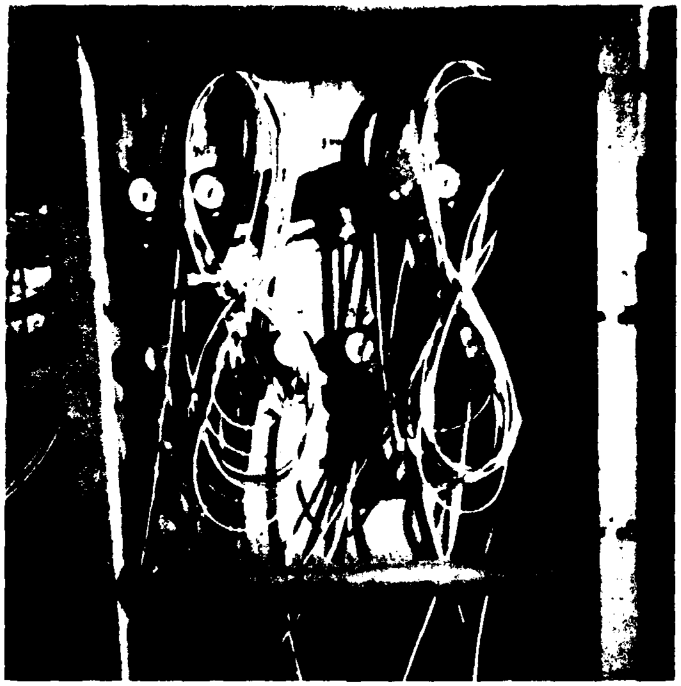


Figure 33. Instrument readout box for WES transducer piezometers and vacuum gages

Settlement plates

61. One settlement plate to measure settlement of the underdrainage layer and foundation was installed in each test section. A typical in place settlement plate with riser is shown in Figure 31 (on left). As previously mentioned, these devices, which were carefully plumbed and leveled, also served to support the WES transducer piezometers. A hole was cut in the side of the bridge for the riser pipe and strapping was carefully placed around the riser to help hold it in place, but not bind it. Zero settlement reference elevations for these devices are given in the following tabulation.

Test Section	Zero Settlement Reference Elevation (ft msl)
1	21.88
2	21.81
3	21.84
4	23.60
5	23.63

Nuclear moisture-
density meter casing

62. Two casings for the nuclear moisture-density meter were installed about 11 ft apart along the section in each test section (Figure 34). Each casing was carefully plumbed during installation and checked for size with a dummy torpedo to ensure passage of the instrument.

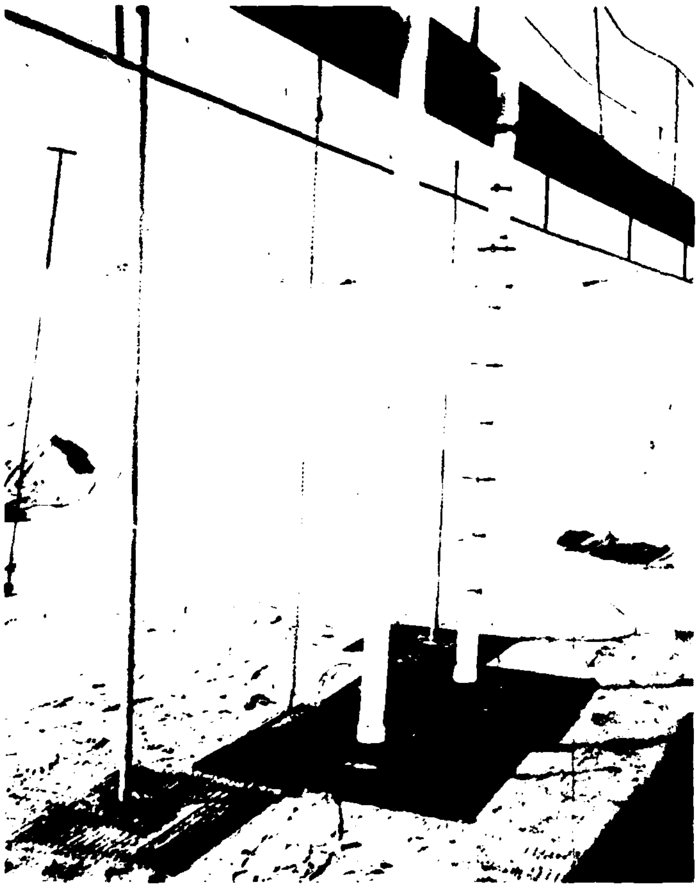


Figure 34. Casings for nuclear
moisture-density meter

Filling of Test Sections with Dredged Material

Dredging

63. Dredging from the borrow area in the existing disposal area

and pumping the material into the test sections was accomplished by a contract with Ed Nemer Construction, Co., Florence, Ala. The dredge used, commonly known as a Mud Cat dredge (Figure 35), was powered by a 175-hp diesel engine, and utilized an 8-in.-diam pipeline for transportation of the dredged material to the test site.

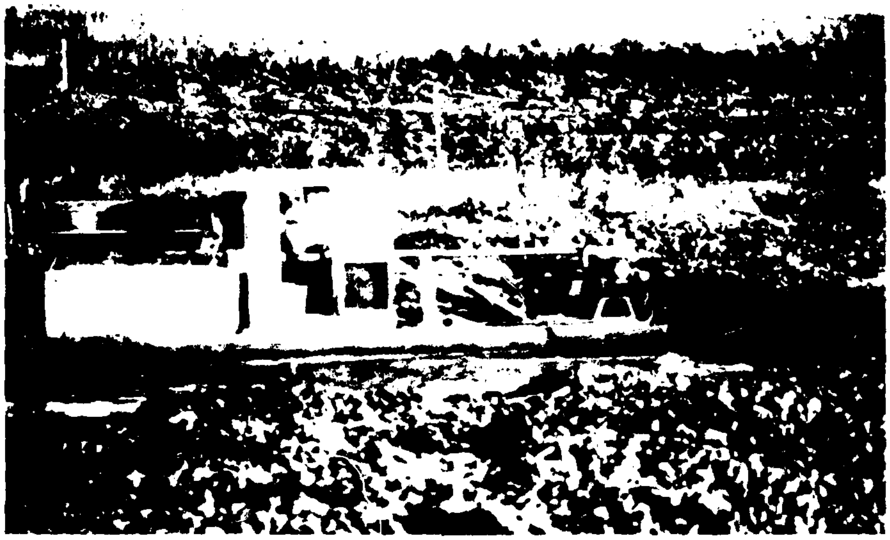


Figure 35. Mud Cat dredge working in borrow area

64. Since a uniform material was desired, it was necessary for the dredge to strip and waste the upper vegetation and stiff crust of material in the borrow area. This allowed the lower, softer material, which more nearly simulated the type of material obtained from prototype maintenance dredging operations, to be utilized in the test sections. This also caused the resulting material in the test sections to be more uniform because the crust material "balled up" considerably and formed a heterogeneous mixture of clay balls and slurry.

Dredge discharge

65. Due to concern that the high exit velocity of the dredged material as it discharged into the test sections would scour the underdrainage layer, an energy dissipator was designed and constructed, and special procedures were used as filling operations began.

66. Energy dissipator. Figure 36 is a photo of the energy dissipator used. The dissipator consisted of a barrel with peripheral slots

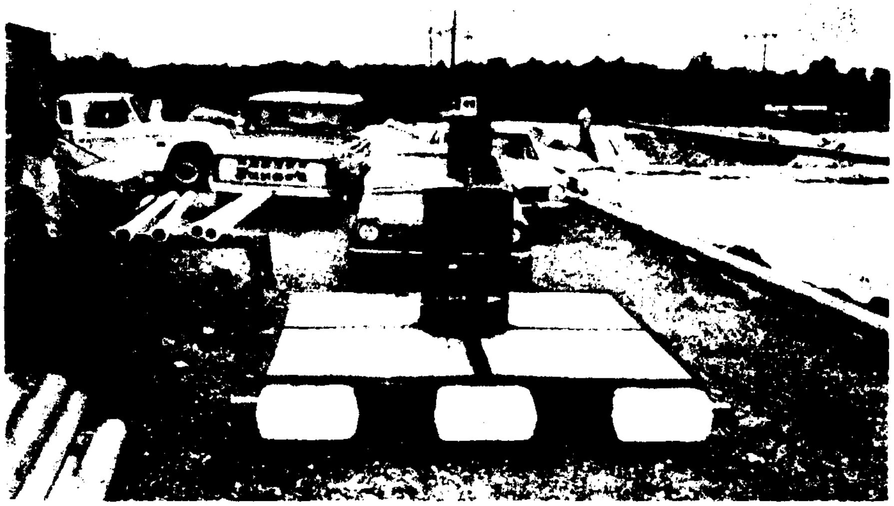


Figure 36. Energy dissipator for dredge discharge pipe

cut in its lower half and mounted on a 4- by 8-ft raft. The raft consisted of a plywood deck mounted in a metal frame with flotation provided by three styrofoam sticks. On top of the barrel a special collar was welded in order to receive the dredge discharge pipe which was provided with a 90-deg elbow on the end. This device worked extremely well, not only in dissipation of the force from the pumped dredged material but also in permitting movement of the discharge within the test section so that a more uniform material resulted.

67. Special procedures during initial filling operations. To further ensure prevention of scour of the underdrainage material a large piece of polypropylene was laid down in the area immediately under the dissipator and water was initially pumped under low operating rpm (Figure 37). The low rpm pumping of water continued until the sand was saturated and the dissipator was barely beginning to float. At that time dredged material was pumped under a slightly higher, but still relatively low, rpm until about 2 ft of slurry was in the section. Dredging was then discontinued and the polypropylene pulled out by a dragline (Figure 38).

Figure 37. Initial filling operations

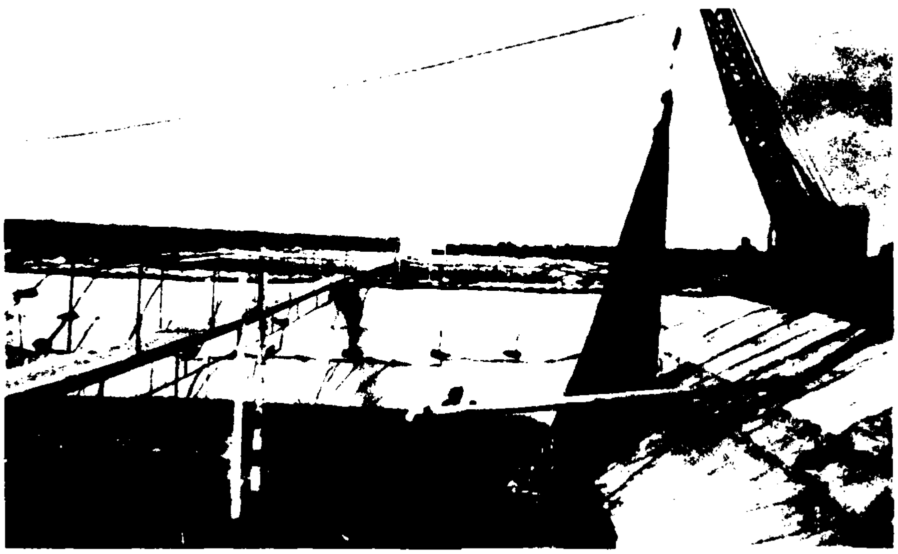


Figure 38. Removal of polypropylene sheeting

Dredging was then resumed with the dredge operating at approximately one-half of its maximum rpm (Figure 39). After about another 2 ft of slurry had been deposited, the dredged was allowed to operated at its maximum rpm (Figure 40).

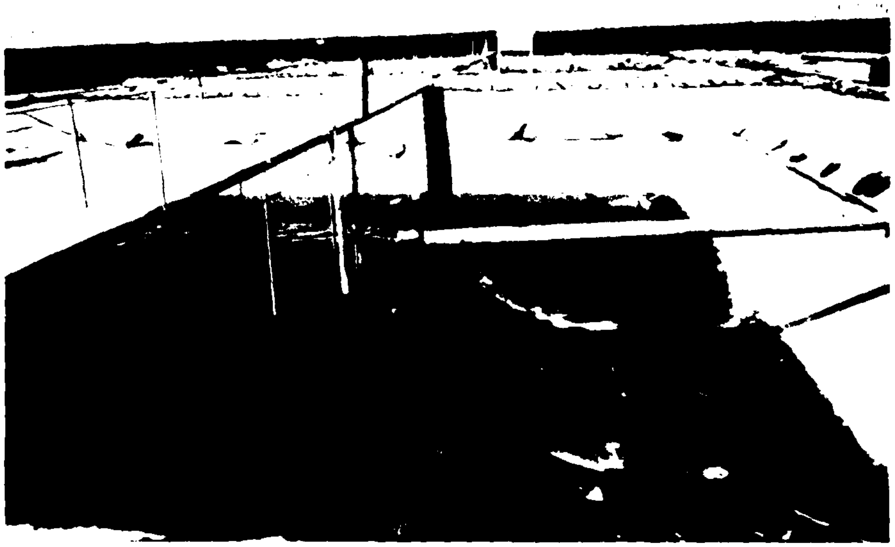


Figure 39. Dredged material discharge with dredge operating at approximately one-half its maximum rpm

Filling sequence

68. Since the slurry being pumped by the dredge contained 15 to 25 percent solids it was necessary to fill each test section several times before the 6-ft desired depth of dredged material was attained. The general procedure for accomplishing this was to pump full, allow the solids to settle out (generally 24 hr was allowed for this), pump the clear water off, and fill again. This procedure was repeated until each test section contained sufficient solids for the experiment. Generally, about $\pm$6.5 ft of dredged material was in each test section when filling was terminated. This overfill allowed for some continued sedimentation which occurred until the material started into the consolidation phase of densification. Figure 41 contains an aerial photo showing the test site after completion of construction.

Figure 40. Dredge discharge with dredge operating at maximum rpm

Figure 41. Completed test site

[illegible] initialization.

70. Dredged material [illegible] test sections 3, 4, and [illegible] to define more accurately the effects of the underdrainage [illegible] desire was to be able to state that all settlement measured [illegible] sections was from consolidation of the material by water being [illegible] from the voids into the underdrainage layer rather than [illegible] or dessication. This was accomplished by installing sumps [illegible] three test sections (3, 4, and 5). Each sump was equipped with [illegible] having automatic on-off floats so that no accumulation of water [illegible] rain would occur; however, the dredged material surface would [illegible] moist by allowing a thin layer of water 1 to 2 in. deep to remain [illegible] surface. The sumps were manually lowered as the dredged material [illegible] face settled. This procedure was followed until most excess pore water had approached hydrostatic and the surface settlements had [illegible] to a steady rate. At this time (about mid-March 1977) the [illegible]

made to allow the dredged material surfaces to dry in order to determine if the densification techniques being evaluated might have an effect on accelerating or increasing the magnitude of surface drying and resultant cracking. To allow this to happen the sumps were lowered to coincide with the dredged material surface. As surface drying began and as cracks were formed, the sumps were periodically lowered to the approximate depth of cracking to prevent accumulation of rainwater.

Maintenance of Ponded Water in Test Sections 1 and 2

71. The original plan was to maintain the ponded water surface in those test sections evaluating seepage consolidation (test sections 1 and 2) at a constant elevation rather than try to maintain a constant load (with respect to the dredged material surface). This was accomplished within a tolerance of about a foot with el 20 being the target elevation. A plot of the water depth over the dredged material surface for the duration of ponding (about 8 months) is shown in Figure 43. The

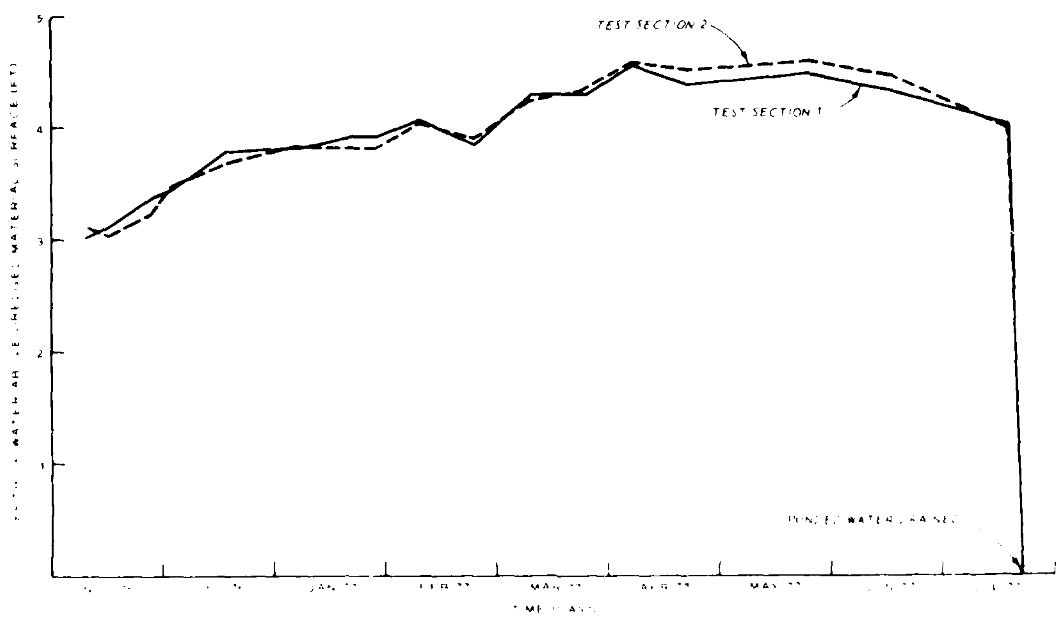


Figure 43. Depth of ponded water vs time for test sections 1 and 2

increase in the water depth from the initial 3 ft was due primarily to settlement of the dredged material surface since the water surface elevation was maintained relatively constant.

Maintenance of Vacuum in Test Sections 2 and 3

72. A plot of the average vacuum maintained in the underdrainage layers of test sections 2 and 3 is shown in Figure 44. As can be seen from this plot, the average vacuum for both test sections gradually diminished with time. This was due somewhat to drying and to the resultant cracking of the dredged material, but for the most part the diminished vacuum was due to the pumps losing efficiency from wear and leaky fittings. The vacuum system was not maintained very well due to a limited budget, and in all fairness the statement must be made that under the circumstances the system performed well. The biggest drawback was that the vacuum system was electrical and often went down from power failures caused by thunderstorms. With proper maintenance and/or perhaps the use of pumps powered by internal combustion engines, it is felt that such a vacuum system would be more practical for prototype application.

Data Collection

73. Instrument readings were initially performed on a daily basis, then weekly, semimonthly, monthly, and finally biomonthly. In situ measurements were taken and sampling was performed on a weekly basis initially, then semimonthly, monthly, and finally bimonthly. All data obtained at the site were reduced, plotted, and checked prior to leaving the site in order to reduce errors.

In situ measurements

74. Vane shear measurements were taken at every foot of depth through the entire thickness of the lift. Each test section was tested in two different places, and the results were averaged to aid in overcoming whatever material nonhomogeneities might have existed.

75. Measurements using the nuclear moisture-density meter to

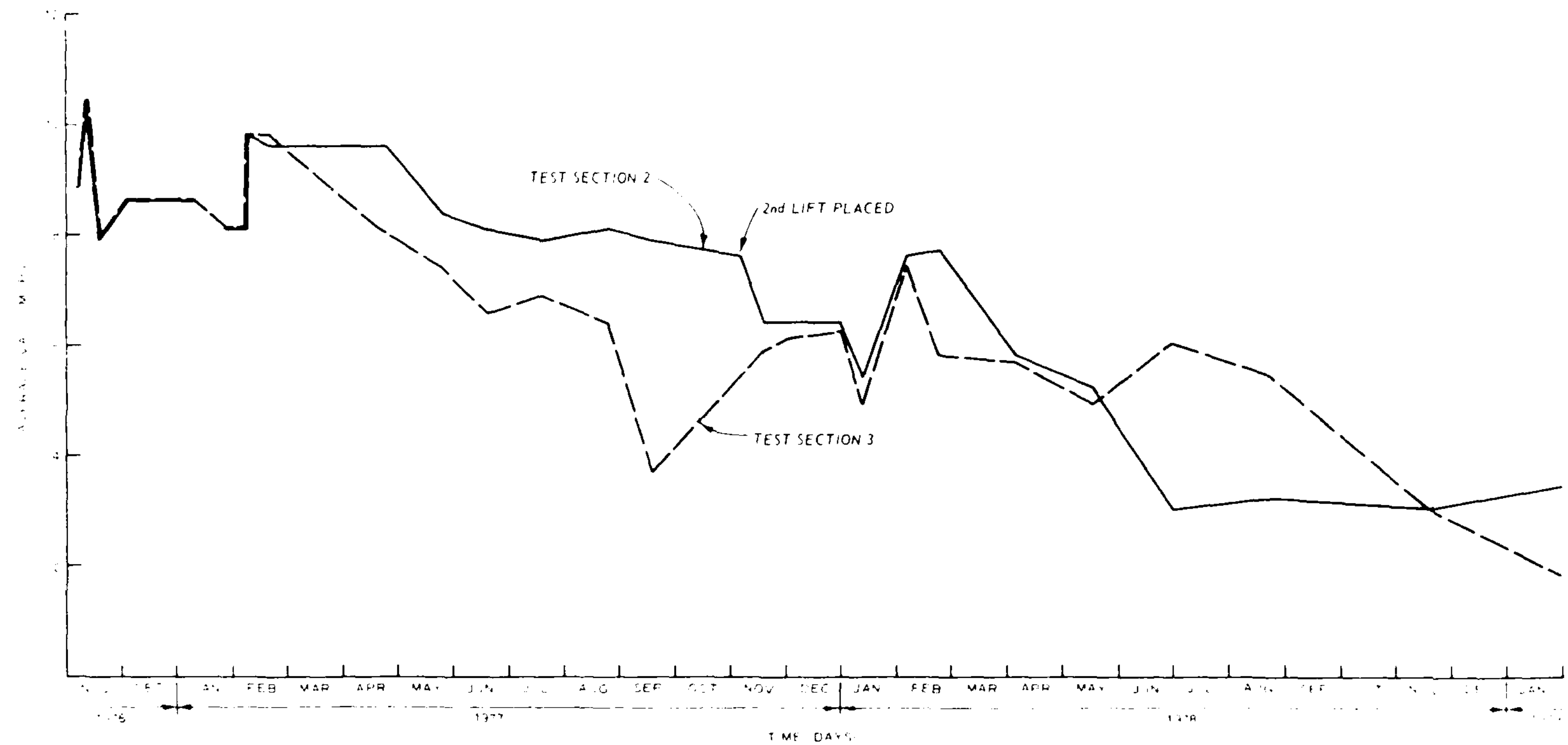


Figure 44. Average vacuum in underdrainage layers vs. time for test sections 2 and 3

determine in situ density and water content were abandoned. Initial readings and one subsequent reading had been taken when the equipment malfunctioned and was sent to the factory for repairs. After approximately one month the equipment was returned and three subsequent sets of readings were obtained. Following an examination of these data, readings were terminated because of data scatter which was of such magnitude as to render the data virtually useless. This may have been due to the organic content present in the solids.

Sampling for water content determinations

76. Like the in situ measurements, samples for water content determinations were taken every foot of depth. Initially the slurry sampler had to be used for all sampling except the bottom sample (i.e., the material immediately overlying the underdrainage layer) which was taken with the Hvorslev sampler. However, as time went on and the material consolidated, the Hvorslev sampler was used at higher and higher elevations until it was used exclusively except for surface samples in the seepage consolidation test sections. This occurred approximately four months into the experiment.

Sampling for index property determinations

77. A complete set of samples was taken in February 1977 and subjected to laboratory testing for Atterberg limits and specific gravity determinations. These samples were taken at 1 ft intervals through each test section.

Sampling for strength determination

78. Three different sets of undisturbed samples of the dredged material immediately overlying the underdrainage layer were taken. The first set was subjected to Q-triaxial tests while the second and third groups were tested in unconfined compression. All samples were taken with the Hvorslev sampler and [illegible] were firm enough to be tested except those from test section 5 (control), which were too soft and slumped badly upon extrusion from the sampler.

Tensiometers

79. No accurate readings could be obtained from the tensiometers due to problems with keeping the system full of de-aired water and with the mercury monometer readout devices. After several attempts the use of these instruments was abandoned.

WES electrical transducers

80. Pore pressure readings from these instruments were erratic and seemed to run consistently on the high side, some impossibly high. Readings were taken throughout the experiment, but due to unexplainable results, they were not used in the analysis.

Porous stone piezometers

81. These instruments continually gave consistent, reliable results. This fact, coupled with the lack of results from the other pore pressure measuring devices, led to their exclusive use for pore pressure data.

82. Because of settlement of the dredged material surface, the piezometers at the 5-ft level became exposed (i.e., were above the dredged material surface) after about two months of operation. In order to have enough data points to define the pore water pressure profile with depth, additional porous stone piezometers were installed at a nominal 1.5-ft level.

PART VI: ADDITION OF SECOND LIFT

Background

83. During the summer of 1977 the U. S. Army Engineer District, Chicago, became interested in this work because they were contemplating an underdrain installation at one of their disposal sites. They were particularly interested in the vacuum application since it seemed to be the best of the four methods being evaluated, and because their project was of such a nature that addition of a vacuum system could be accomplished at relatively low cost. One of the questions they wanted answered was: Could vacuum-assisted underdrainage be effective on more than one lift of dredged material; if so, what measures would have to be taken to ensure its success? Therefore, at the request and funding of the Chicago District, this study was extended for another year in order to facilitate evaluation of the different techniques for a second 6-ft lift of dredged material.

Design

Configuration

84. The following scheme of study for the second lift of dredged material was agreed upon by the Chicago District and WES.

85. Test section 2 (formerly vacuum-assisted seepage consolidation). A 1-ft-thick sand layer (hereinafter referred to as an intermediate drainage layer) would be placed on top of the first lift and connected to the original underdrainage layer by four 8-in.-diam vertical sand columns (Figure 45). The purpose of this configuration was to see if the vacuum in the original underdrainage layer could propagate through the sand columns into the intermediate drainage layer thereby providing a vacuum at the base of the second lift. Also, by connecting the two sand layers, an exit would be provided for water collecting in the intermediate layer.

86. Test section 3 (partial vacuum in underdrainage layer). The first lift in this test section had undergone the most extensive cracking

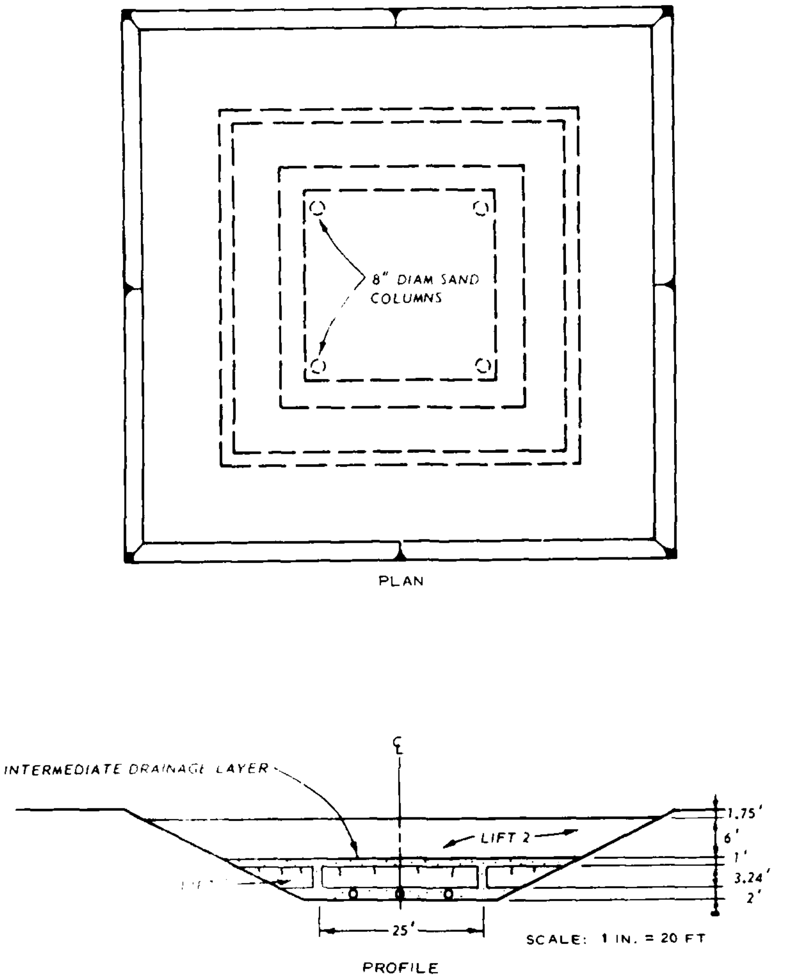


Figure 45. Test section 2 plan and profile (second lift)

and had, in fact, several cracks extending through its entire depth. It was therefore decided to again place a 1-ft intermediate drainage layer between the two lifts, but without the sand columns planned for test section 2. Instead, the sand would be flooded after placement in order to facilitate filling of the existing cracks in the first lift with sand, thereby providing a connection between the two drainage layers (Figure 46).

87. Test section 4 (gravity underdrainage). No special provisions would be made for this test section, i.e., a second lift would be pumped

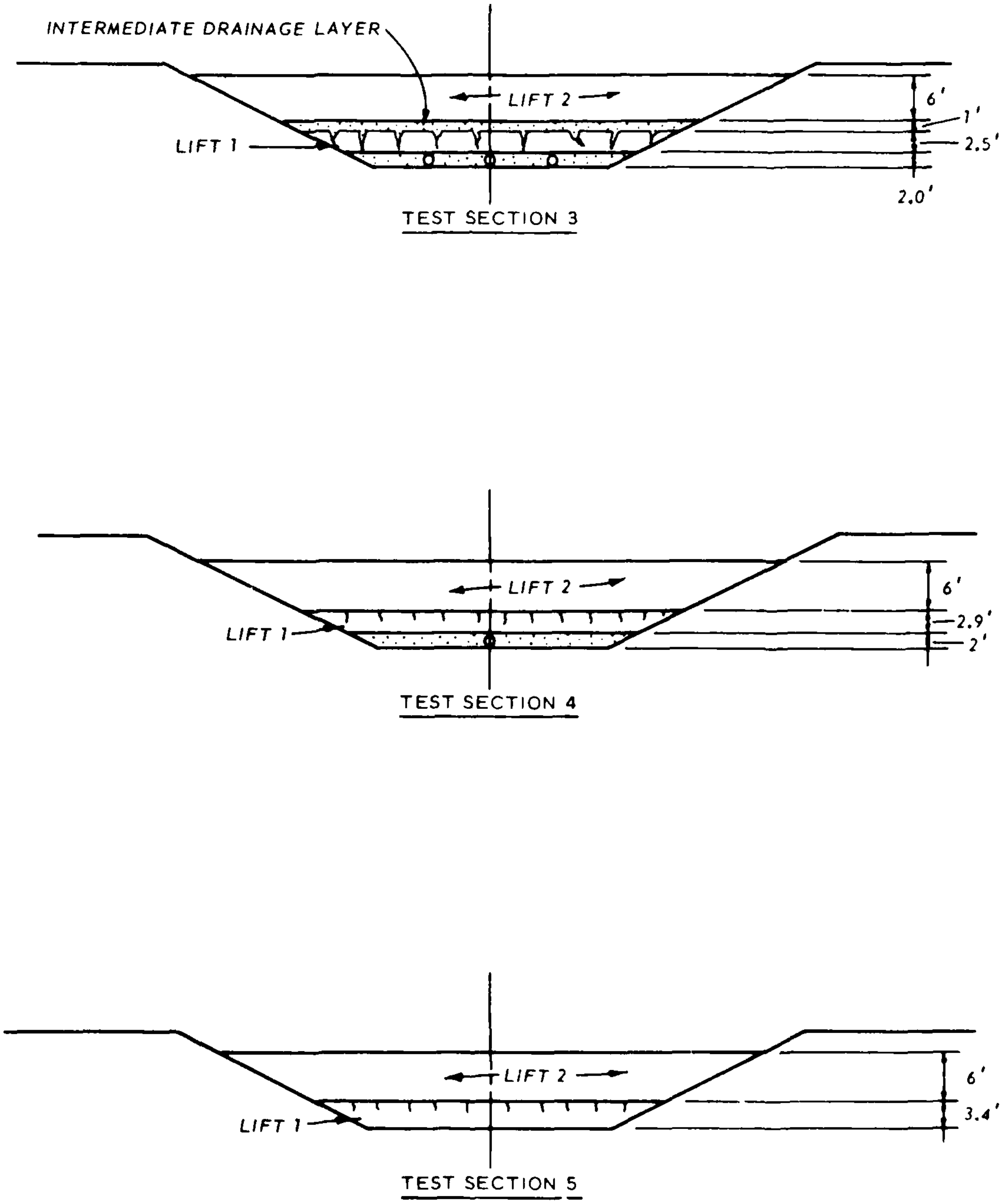


Figure 46. Profiles - test sections 3, 4, and 5

in on top of the first (Figure 46). Essentially, this would determine if the original underdrainage layer would have any effect on a second lift, merely by gravity drainage.

88. Test section 5 (control). As was the case with test section 4, a second lift would be pumped directly on the first lift as shown in Figure 46. This section would provide a non-treatment base against which the effects of the treated sections could be measured.

Site preparation

89. In order to accomplish the preceeding plans some additional site preparation would be necessary. The site grade would have to be raised, mostly at the end near test section 5 and none at test section 1, which would not receive a second lift. After surveying, a 3-ft grade raise was required at test section 5, which could taper to zero at test section 2. Also involved would be raising the access bridges an appropriate amount (i.e. 3 ft for test section 5, 2 ft for test section 4, and 1 ft for test section 3). This could be accomplished easiest by using timber cribbing at the bridge abutments.

90. The clear polypropylene plastic used originally to line the pits had deteriorated badly above the dredged material (i.e., where exposed) due to the effects of sunlight and wind. Because of this, and because of raising the grade, the pits would have to be relined above the level of lift one. It was decided to use a thicker, black plastic, reinforced with nylon fibers for relining.

Instrumentation

91. Based on the performance of instruments during monitoring of the first lift, the WES transducers and the tensiometers were eliminated and only porous-stone-type piezometers would be installed for pore pressure measurements in the second lift of dredged material. The slotted pore-pipe-type instrument used to measure vacuum in the underdrainage layer would be installed in the intermediate drainage layer for vacuum measurements.

92. Settlement plates similar to those installed previously for measurement of foundation settlement would be installed on the surface of lift 1 to allow monitoring of the settlement of lift 1 after placement

of the second lift. Surface readings would be taken on lift 2 to monitor its settlement.

Construction

Site preparation

93. The site grade was raised as planned by casting sand up with a dragline and blading with a small bulldozer. Following this operation, cribbing consisting of 6- by 6-in. timbers was used to raise the access bridges for test sections 3, 4, and 5. This operation consisted of raising one end of a bridge with a crane, placing the cribbing, and setting the bridge back into place with the crane. This operation was then duplicated on the other end of the bridge.

94. The next step was to place the black, nylon-reinforced liners. This was accomplished by lining one side of a test section at a time, lapping over, and using a special tape to tie the pieces together. On the surface of the first lift, 5 ft of liner was laid down and sandbagged in test sections 4 and 5 while the intermediate drainage was placed over the liner in test sections 2 and 3. At the top of all sections a sand ridge was bladed up, the liner laid over it, and the backside covered with sand to provide anchorage from wind. Typical liner installations are shown in Figure 47.

Intermediate drainage layers

95. In test section 2 the sand columns were placed first by driving four 8-in.-diam steel pipes through the first lift into the underdrainage layer and the material was excavated from within using posthole diggers and by jetting. When excavation was complete a distinct hissing from the existing vacuum could be heard. At that point the pipes were filled with sand and flooded to ensure a good connection and facilitate compaction of the sand. The steel pipes were left in place since the sand columns were "connectors" only and were not intended to serve as vertical sand drains for the first lift. A vacuum measuring instrument was then placed in each of the four columns. Then the intermediate drainage layers for sections 2 and 3 were placed by clamshell and spread to the desired 1-ft thickness by hand.

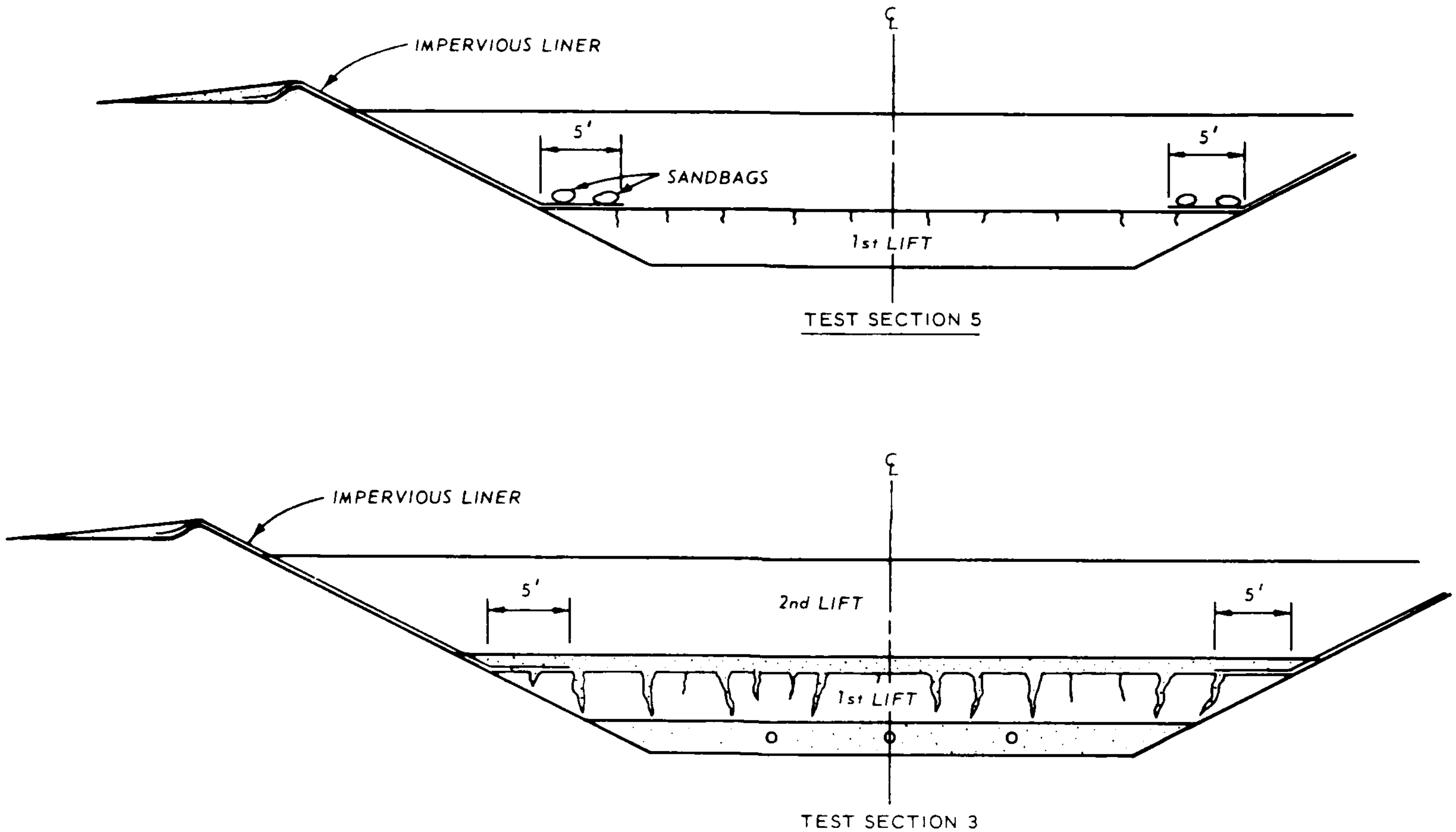


Figure 47. Typical installation of impervious liner for second lift

Instrumentation

96. Porous-stone-type piezometers wrapped in sand-filled bags (the same piezometers as used for the first lift) were placed on instrument stands and hung at nominal 1-ft levels for the second lift. Settlement plates of the same order as those used for the first lift were placed directly on the surface of the first lift in sections 4 and 5. The same type of plates were placed on the top of the intermediate drainage layer in test sections 2 and 3 (the thickness of the sand layer being carefully measured under the plates prior to placement). Three lines of vacuum measuring devices (one on the centerline and one 2.5 ft either side of the centerline) containing three devices each (nine total) were then placed in the intermediate drainage layers in test sections 2 and 3. Each device was placed between the bottom and the middle of the layer.

Pumping of second lift

97. Exactly the same procedures and equipment used to place the first lift were used to pump the second lift with one exception. There were no sand layers to protect in test sections 4 and 5 so no special procedures were required in these sections for protection of the sand. Pumping began on 30 October 1977 and was completed on 22 November 1977. A photo of the site upon completion of pumping is shown in Figure 48. During placement of the second lift all drainage systems were left open and vacuum pumps left on. Instrument readings were taken during pumping.

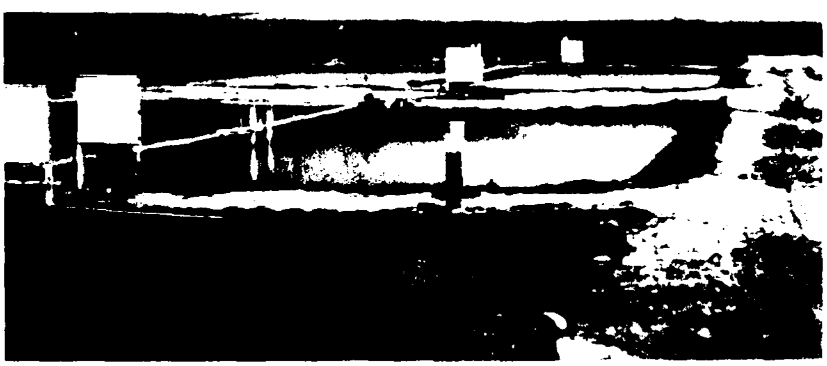


Figure 48. Test site upon completion of second lift placement

Conduct of Experiment

Control of surface water

98. Sumps were installed in the second lift of all four sections. The sumps were set to remove all surface water thus allowing surface drying. However, there was so much rain during the winter that very little drying occurred. The sumps did prevent ponding of rainwater during the winter months. It was only in the early summer of 1978 that dessication cracks began to show on the surface. However, frequent power failures during the summer rendered the sump pumps inoperative, which allowed the ponding of surface water. Thus the effects of drying were not as pronounced as would have been the case had the pumps been operating constantly.

Maintenance of vacuum in test sections 2 and 3

99. Maximum readings from vacuum measuring devices in the intermediate drainage layer were about 1 psi and occurred in one instrument (out of nine) in test section 3 and two instruments (out of nine) in test section 2. All of the remaining gages read zero in both test sections throughout the study. However, a hiss could be heard from these gages when the valves were sharply cracked, indicating a pressure less than atmospheric did exist throughout the intermediate drainage layer of both test sections.

Intrumentation

100. All porous-stone-type piezometers were read on an approximate monthly basis in both lifts and appeared to give reasonable readings. Settlement plates were also read on a monthly basis along with surveys of the second lift surface.

In situ testing and sampling

101. In situ vane shear tests and sampling for water content determinations were also performed on a monthly basis. Both lifts were tested and sampled in test sections 4 and 5, but only the second lift was tested and sampled in test sections 2 and 3 since the intermediate sand layer prevented penetration by both the vane shear device and the sampler.

PART VIII: PRESENTATION AND DISCUSSION OF TEST DATA

Settlement

102. The net settlement of each dredged material lift was computed from lift surface surveys and settlement plate measurements. Net settlement for the first lift consists of gross settlement minus foundation settlement. Net settlement for the second lift consists of gross second lift settlement minus gross first lift settlement (which includes foundation settlement). Since original lift thicknesses varied somewhat, it was necessary to normalize the settlement data with respect to original layer thickness. Results are therefore presented as percent strain versus time, computed as follows:

$$\%\ \text{strain} = \Delta H / H_i \times 100$$

where

ΔH = net settlement, ft

H_i = original layer thickness, ft

Percent strain values versus time for lift 1 are presented in Figure 49 and in Figure 50 for lift 2.

Lift 1

103. Several observations can be made from examination of Figure 49; these are:

- a. All treated sections settled more in the time frame of the experiment than did the control (untreated) section.
- b. Benefits gained by underdrainage essentially all occurred during the first 160 days.
- c. The rate at which additional settlements occurred in test sections 1 and 2 (seepage consolidation) was different than those for test sections 3 and 4.
- d. The majority of the total settlement measured in test sections 2, 3, and 4 occurred prior to placing the second lift (i.e. approximately 1 year) while test sections 1 (no second lift) and 5 (control) continued to settle after this time.

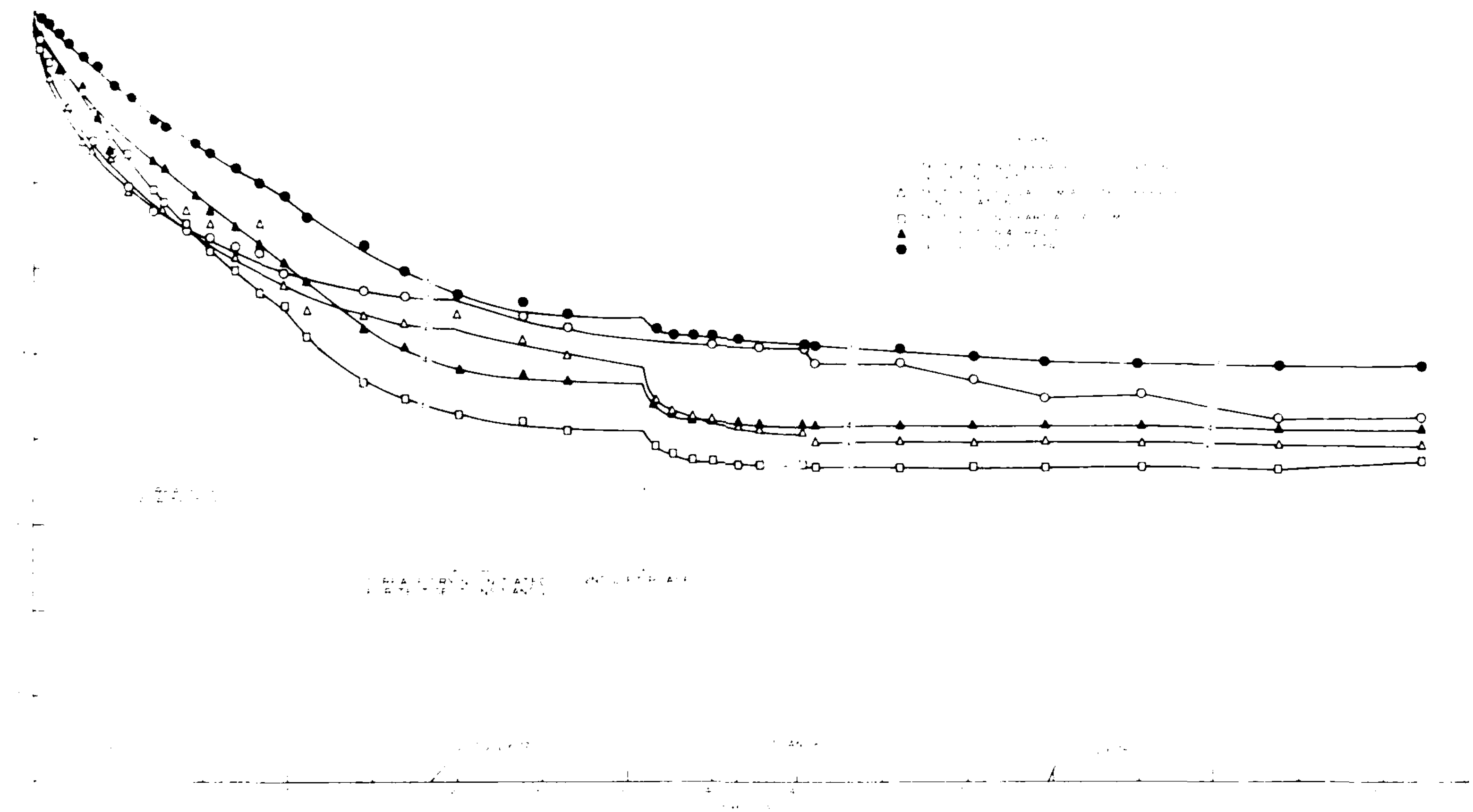


Figure 49. First lift settlement vs time

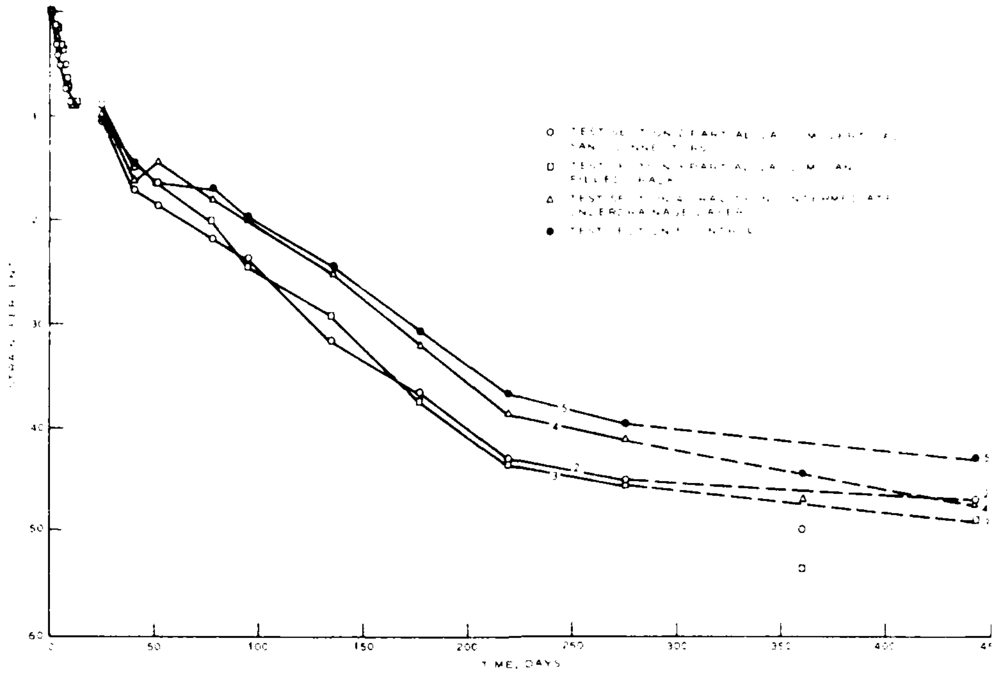


Figure 50. Second lift settlement vs time

The following paragraphs contain a discussion of these observations.

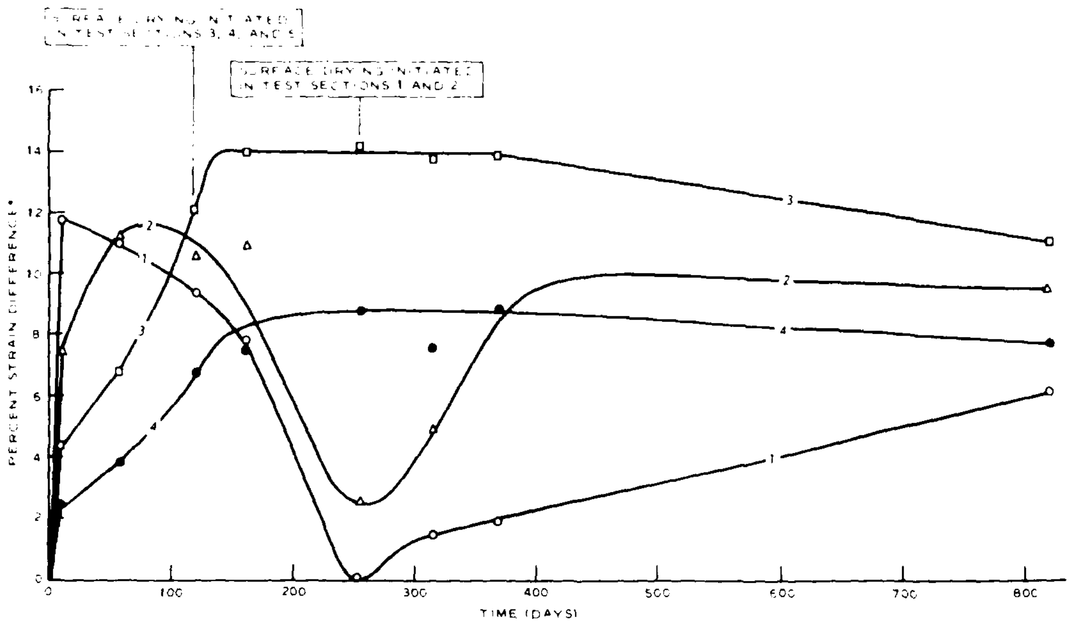
104. The final differences in settlement (measured at the end of the experiment) between the treated sections and the control section were actually reached about 160 days after the experiment and then remained essentially constant or decreased slightly for the remainder of the experiment. Only the percent strain curves for test sections 1 and 5 show continued settlement in lift 1 after placement of lift 2 (i.e. in the later stages of the experiment). This is because test section 1 received no second lift and was therefore subjected to continued surface drying while the first lifts of test sections 2 through 5 (which did receive second lifts) were not. The first lift of test section 5 continues to settle because it is settling at a slower rate and may eventually equal the total settlement of the treated sections, although it would probably take several years. Also, since less settlement had occurred in test section 5 prior to surcharging (placement of the second

lift), it would seem likely the surcharging would have a greater effect on test section 5 than the others.

105. The fact that all benefits occurred early in the experiment is most likely explained by the fact that the majority of initial consolidation occurs in the material closest to the drainage face; as this occurs, the material becomes denser and its permeability lower thus inhibiting the consolidation of material above it. This would account for a very fast initial consolidation rate, which would decrease fairly rapidly. Other data subsequently discussed confirm this.

106. The settlement curves show initial settlements in test sections 1 and 2, which were subjected to a 3-ft ponding of water, occurred at a faster rate than settlements in the other three test sections. However, about 90 days after experiment initialization the settlement rates in test sections 1 and 2 had decreased, as evidenced by the slope of their settlement curves becoming flatter while test sections 3, 4, and 5 continued to settle at a linear rate up to about 200 days. A possible explanation of this observation is: In the early stages of the experiment, the increased hydraulic gradient existing in test sections 1 and 2 contributed more to consolidation of the dredged material than did the partial vacuum or gravity underdrainage systems in test sections 3 and 4, respectively. Also, no surface drying was permitted in test sections 3, 4, and 5 during this period. After surface drying began in test sections 3 and 4 but still was prohibited in test sections 1 and 2 due to the ponded water, the curves for 3 and 4 continued downward surpassing those for 1 and 2, which were leveling off.

107. Figure 51 confirms the fact that most benefits from the methods evaluated in test sections 1-4 occurred very early, up to about 90 days after beginning the experiment for test sections 1 and 2 and up to about 140 days for test sections 3 and 4. The ordinate of the plot in Figure 51 is the difference between the percent strain of test sections 1-4 and the percent strain of test section 5. This difference was maximized at about 140 days for test sections 3 and 4 and then remained essentially constant for the remainder of the experiment. Test sections 3 and 4 are directly comparable to test section 5 in this manner



* DIFFERENCE BETWEEN PERCENT STRAIN OF ANY PARTICULAR TEST SECTION AND PERCENT STRAIN OF THE CONTROL TEST SECTION (5).

Figure 51. Percent strain difference versus time for lift 1

as surface drying was identical for all three. Even though prohibition of surface drying in test sections 1 and 2 for an additional 135 days makes a direct comparison of test sections 1 and 2 with 5 inappropriate, the curves in Figure 51 do deserve some attention. During the time surface drying was occurring in test section 5 but not sections 1 and 2, the increased settlements in section 5 nearly equaled those that had occurred earlier in sections 1 and 2 (this occurred at about 250 days). Test sections 1 and 2 settlements increased steadily immediately upon initialization of surface drying until they had almost regained their earlier lead over test section 5. This would indicate that volume decrease due to surface dessication occurs at a decaying rate as the surface dries and evaporation begins to slow due to a lesser amount of water in the material being exposed to direct sunlight and wind.

108. Table 8 summarizes the first lift settlements for all test sections at two different points in time: (a) after approximately 1 year (just prior to placing lift 2) and (b) after approximately 2 years (the end of the experiment). Based on the summary prior to

placement of lift 2, test section 3 (partial vacuum in underdrainage layer) experienced the most settlement followed by test section 4 (gravity underdrainage), then test section 2 (vacuum-assisted seepage consolidation), and lastly test section 1 (seepage consolidation). Test section 3 still showed the most percent settlement at the end of the experiment, but test section 2 moved up to second place followed by test sections 4 and 1 (which had no surcharge). This would indicate the vacuum used in test section 2 began to have an affect over this period of time indicating the vacuum is more effective when used independently of the ponded water. The effectiveness of the vacuum is further borne out by the results of test section 3. Also, the surcharge of lift 2 on lift 1 probably had a greater affect on test section 2 because at the time of its placement test section 2 had not undergone as much settlement as test section 4.

Lift 2

109. Lift 2 settlement curves in Figure 50 indicate basically the same thing as did those for lift 1. The treated test sections (2, 3, and 4) all settled more than did the untreated section (5), and the gain was experienced very early after placement. However, benefits realized from the treatments were not nearly as great as were those realized on lift 1.

110. After early benefits were realized (up to about 90 days), all the curves indicate settlements in all four sections occurred at a similar rate and began to decrease (also in a similar fashion) between 200 and 250 days. This phenomenon occurred somewhat later in the first lift (at about 300 days), perhaps because surface drying was initially prohibited for the first lift while for the second it was not.

111. Table 9 presents a summary of lift 2 settlements. Test section 3 again experienced the most settlement followed closely by test sections 4 and 2 which, for all practical purposes, experienced the same amount of settlement. As noted earlier, the magnitude of the percent increase of the treated sections was considerably less for lift 2 than for lift 1 even though the percent settlements themselves were about the same as those for the first lift. Further inspection reveals that the

percent settlement of the control test section (5) is substantially higher than for the first lift, thus causing the percent increases for the treated sections to be lower.

Water Content

112. Water content determinations were made on a routine basis throughout the experiment with sampling generally taking place every foot of depth. However, lift 1 was not sampled after placement of lift 2. Percent water content was calculated as follows:

$$w = \frac{W_w}{Ws} \times 100$$

where

w = water content, percent

W_w = weight of water, g

Ws = weight of solids, g

Lift 1

113. Initial (November 1976) and final (September 1977) water contents for the first lift of all five test sections are shown plotted with depth in Figures 52-56. These plots indicate that considerable dewatering of the dredged material took place in all five test sections. It is interesting to note that the initial water content profile for all test sections is roughly linear, while the final profile is more parabolic in shape. The linearity of the initial water content profile is due to somewhat coarser material being located nearer the bottom (caused by the coarser particles settling out faster during pumping), and because the load increases with depth. The much lower initial water contents for the material located adjacent to the underdrainage layer indicate that some consolidation did take place in the 19 days between filling and experiment initialization, probably aided in some sections by leaky valves on the discharge pipes.

114. The parabolic shape of the final water content profiles is typical of a deposit undergoing consolidation with double drainage

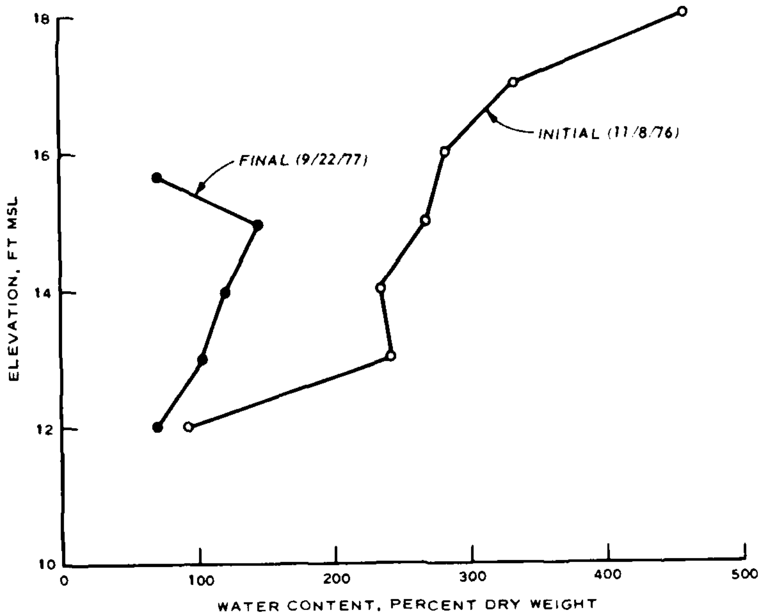


Figure 52. Initial and final water contents, lift 1, test section 1 (seepage consolidation)

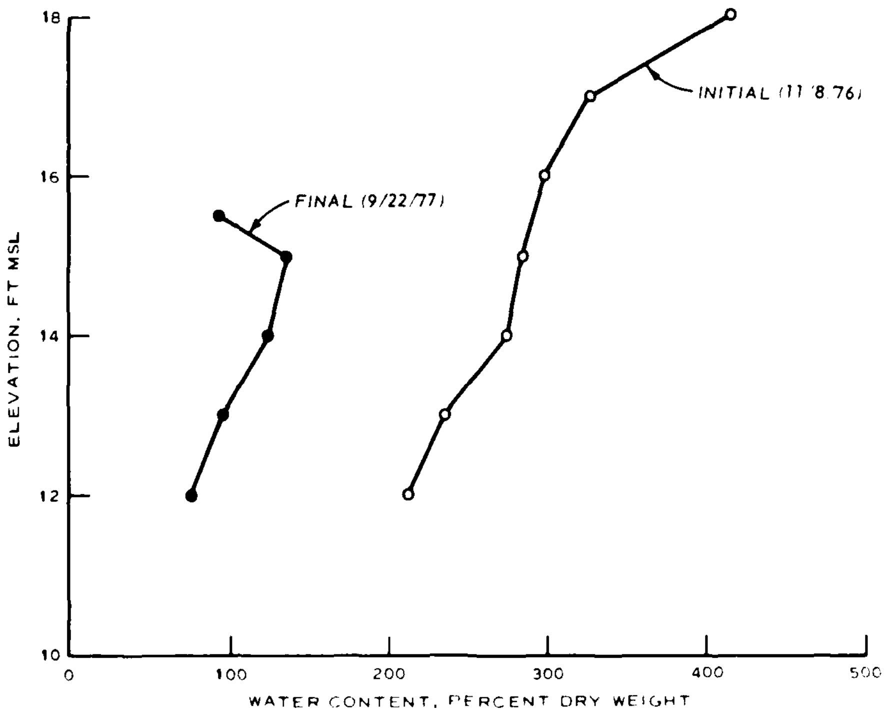


Figure 53. Initial and final water contents, lift 1, test section 2 (vacuum assisted seepage consolidation)

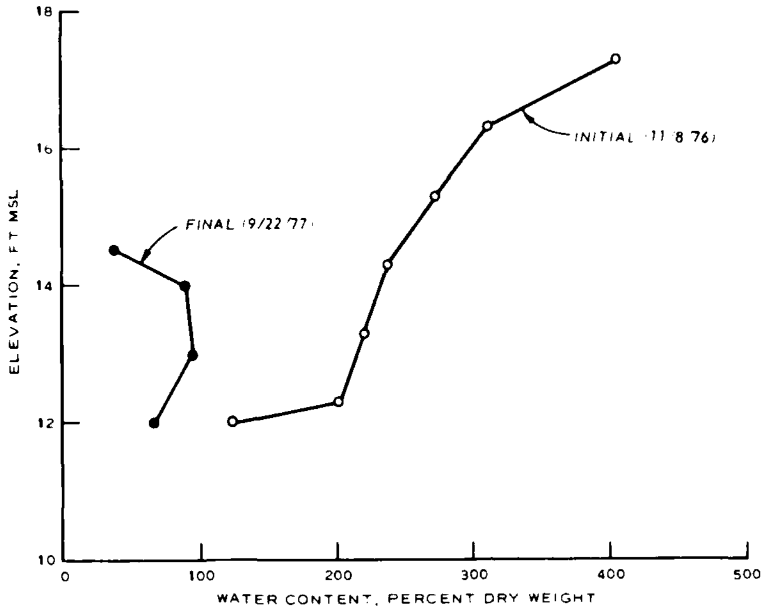


Figure 54. Initial and final water contents, lift 1, test section 3 (partial vacuum)

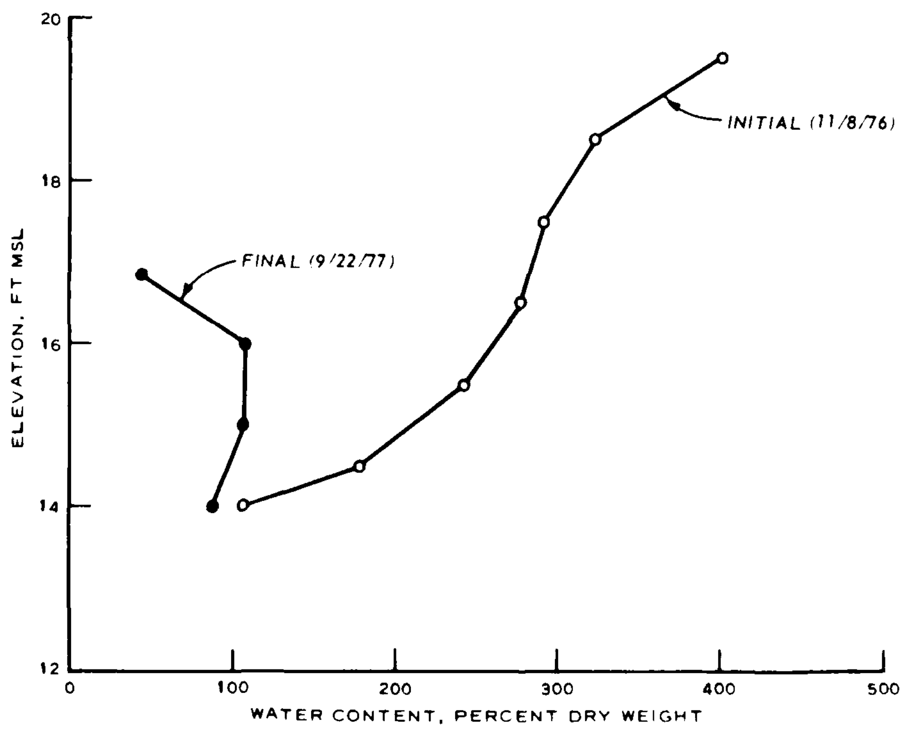


Figure 55. Initial and final water contents, lift 1, test section 4 (gravity)

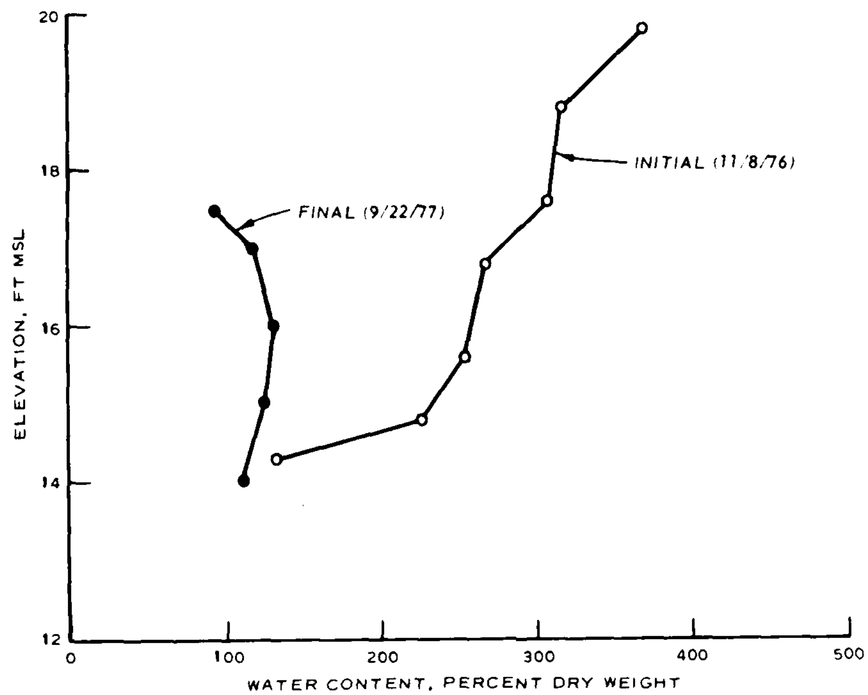


Figure 56. Initial and final water contents, lift 1, test section 5 (control)

(i.e., drainage taking place both in the top and bottom of the layer). Of course, consolidation in the top was aided by dessication. This means the highest water content material is found in the middle of the layer, the furtherest point from the drainage faces. With respect to this, it is interesting to note that for test sections 1 and 2, which did not receive as much surface drying as sections 3, 4, and 5, the highest final water contents were nearer the surface, thus reflecting the effects of surface dessication.

115. Figure 57 contains a plot of final water content profiles for all five test sections. This plot permits comparison between the treated sections and the untreated or control section and comparison between the treated sections themselves. Figure 57 indicates that all treated sections had lower final water contents than did the untreated control section. It also shows that test section 3 (partial vacuum) underwent the most water content reduction followed by test section 4

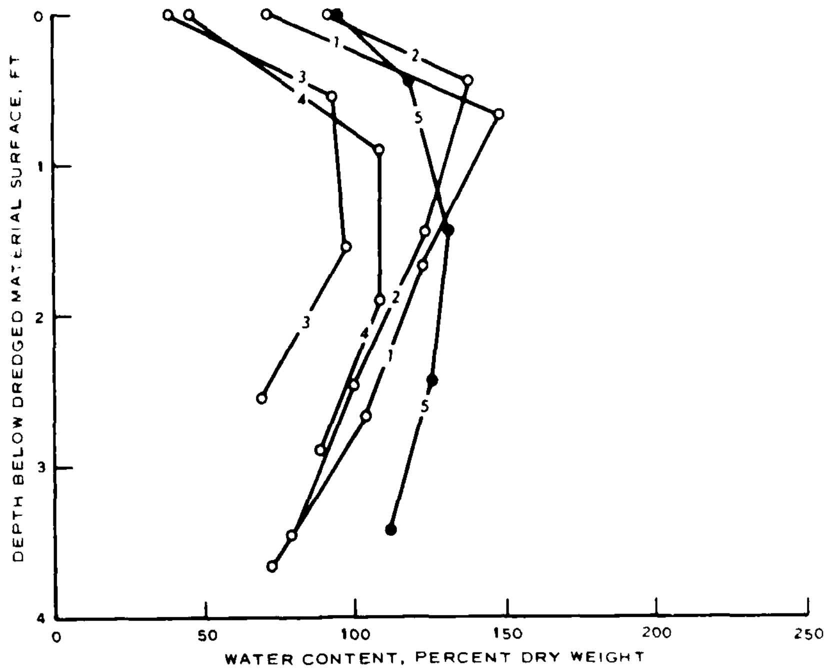


Figure 57. Final water contents, lift 1, test sections 1 through 5

(gravity), test section 2 (vacuum-assisted seepage consolidation), and finally test section 1 (seepage consolidation).

116. Figure 58 shows the change in water content from initial to final, expressed as a percent of the initial water content. This plot normalizes the data with respect to initial water content and essentially yields the same results as Figure 57 except that the lowermost point of test section 2 indicates the greatest percent reduction of all test sections at this point. This value is due to an abnormally high value of initial water content, which probably is not a true indication of the initial water content for the material adjacent to the drainage face in test section 2.

117. The water content data for lift 1 are consistent with the settlement data given in Table 8 for 321 days. No comparison can be made with the 819-day data in Table 8 since lift 1 was not sampled after 321 days.

$$\Delta w = \frac{w_i - w_f}{w_i} \times 100$$

WHERE

Δw = CHANGES IN WATER CONTENT

w_i = INITIAL WATER CONTENT

w_f = FINAL WATER CONTENT

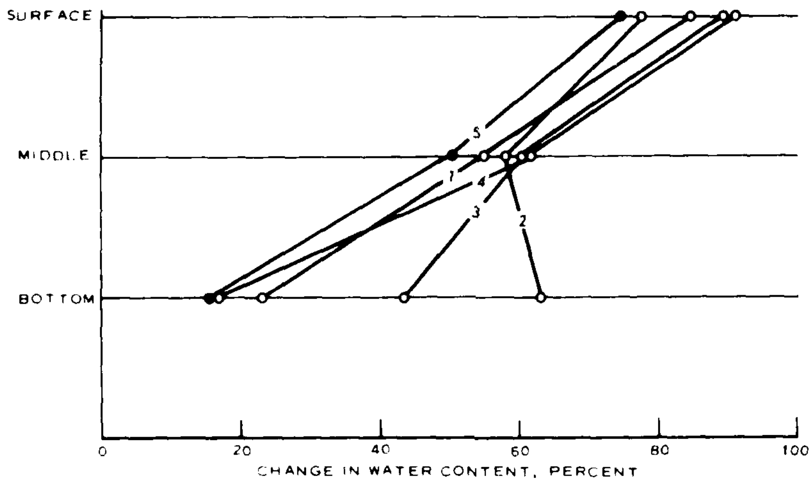


Figure 58. Change in water content, lift 1, test sections 1 through 5

Lift 2

118. Initial (November 1977) and final (February 1979) water contents are shown plotted with depth in Figures 59-62. As with lift 1, these plots indicate that considerable dewatering of the material in lift 2 took place in all four test sections (test section 1 did not have a second lift) with the initial and final profiles also showing linear and parabolic shapes, respectively. The final profiles for test sections 4 and 5 do not exhibit as parabolic a shape as do those for test sections 2 and 3. This is because little additional drying is indicated at the lift bottom over that indicated in the middle, thus resulting in a more vertical profile from the middle to the bottom. This is clear evidence of the benefit of having a drainage layer at the base of the lift (test sections 2 and 3 having one while 4 and 5 did not). It is not readily evident why the final profile for test section 4 did not reflect the effects of surface drying as it was subjected to the same exposure as the other three test sections.

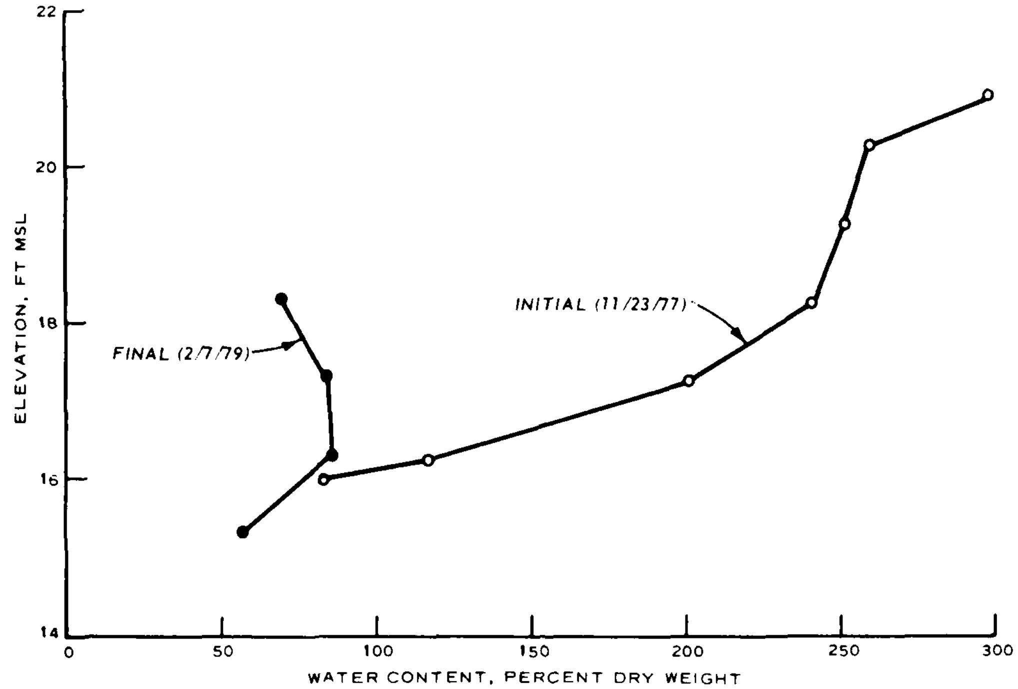


Figure 59. Initial and final water contents, lift 2, test section 2 (vacuum assisted seepage consolidation)

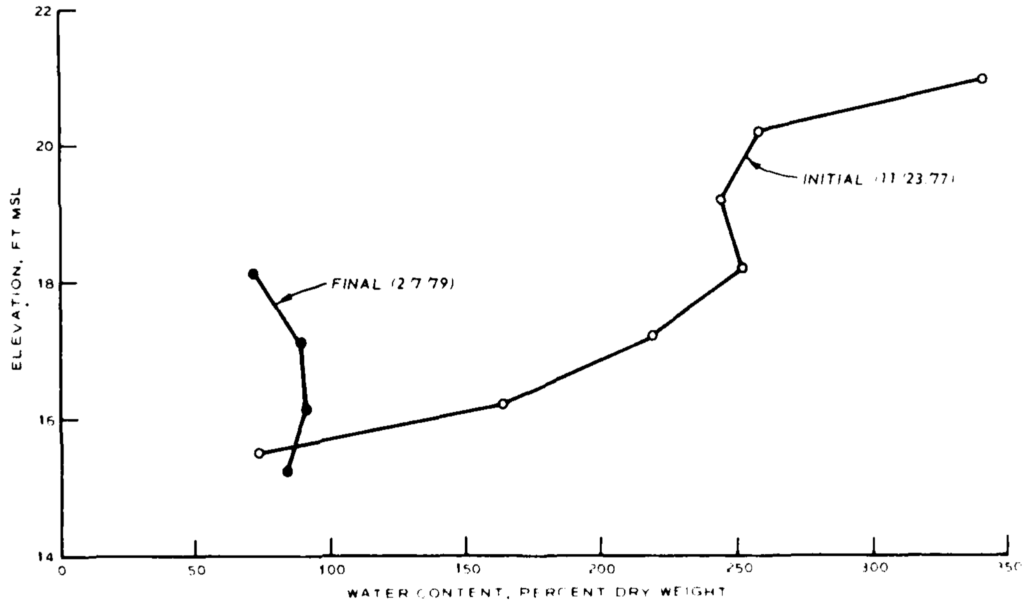


Figure 60. Initial and final water contents, lift 2, test section 3

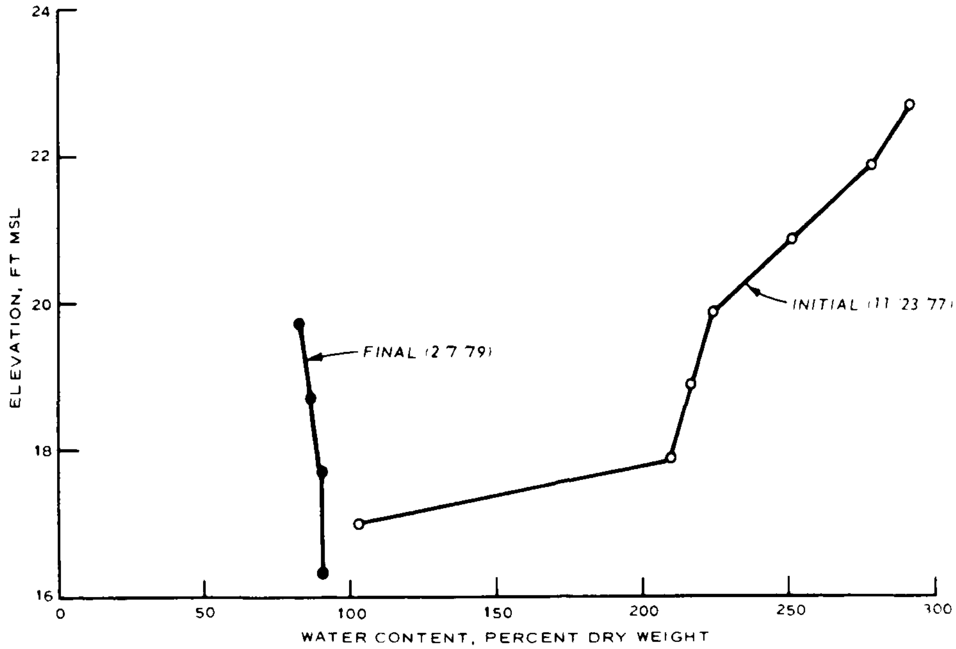


Figure 61. Initial and final water contents, lift 2, test section 4

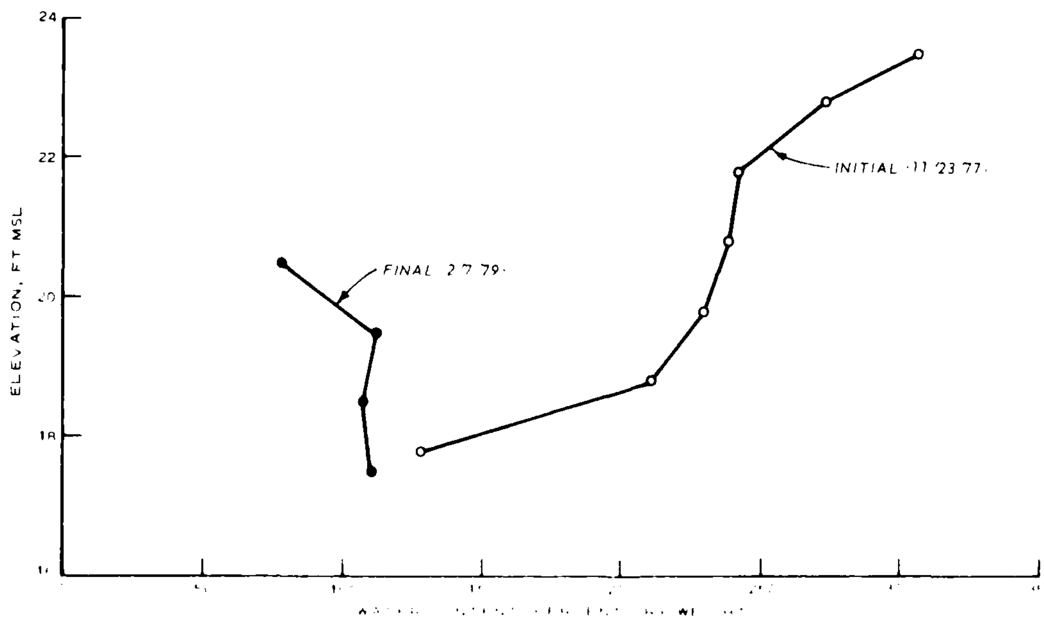


Figure 62. Initial and final water contents, lift 2, test section 5

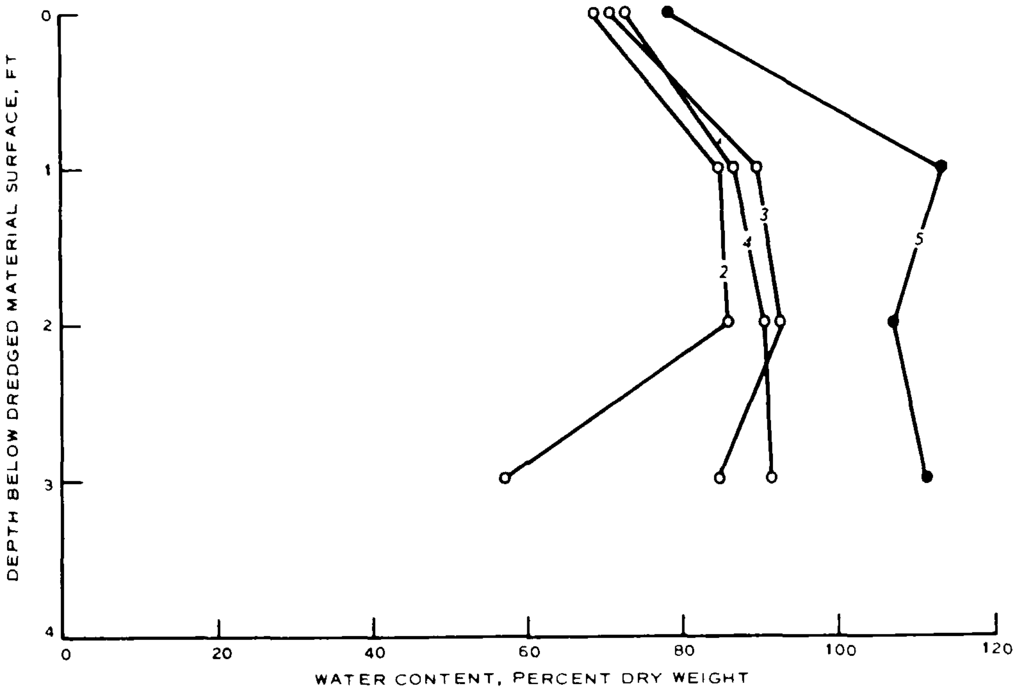


Figure 63. Final water contents, lift 2, test sections 2-5

119. Figure 63 contains a plot of final water content profiles for all four test sections. This plot also shows that all treated sections dried more than the untreated or control section. Test section 2 (with the intermediate drainage layer connected to the lower drainage layer) exhibited the most drying followed by test section 4 (gravity, no intermediate drainage layer), and finally test section 3 (unconnected, except for cracks, intermediate drainage layer). While test section 2 stands out as having dewatered the most according to these data, there is little difference between test sections 3 and 4.

Pore Pressure

Lift 1

120. Plots of initial and final pore pressure distribution for all five test sections are presented in Figures 64-68. Initial refers to just prior to initiating drainage while final readings were taken prior to placing the second lift. Each plot also contains the hydrostatic

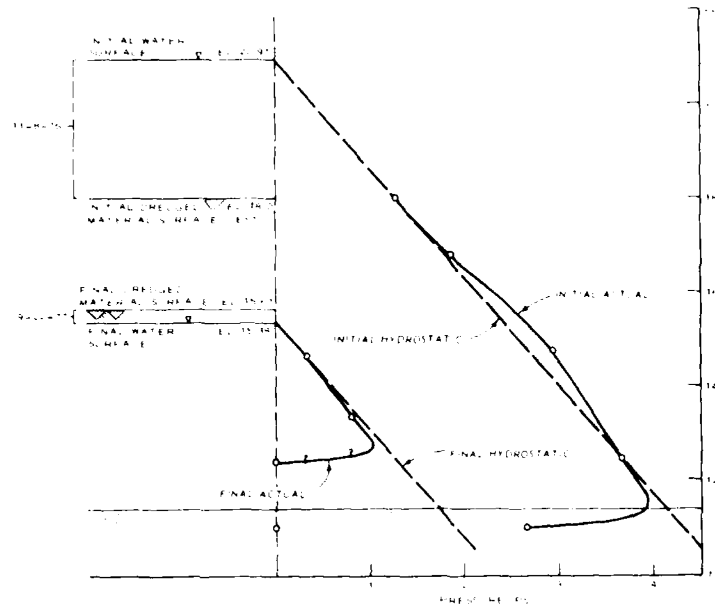


Figure 64. Initial and final pore pressure distribution, test section 1 (seepage consolidation) lift 1

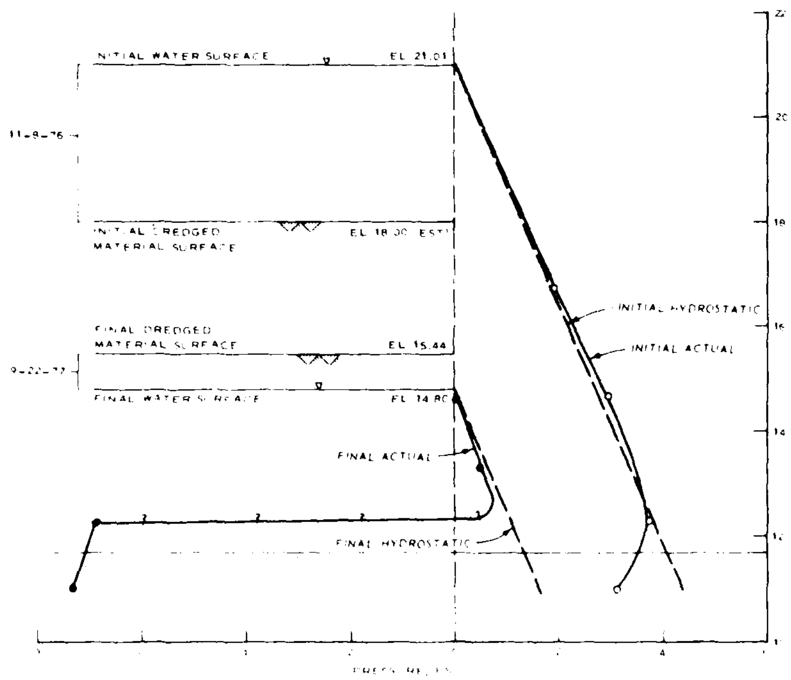


Figure 65. Initial and final pore pressure distribution, test section 2 (vacuum assisted seepage consolidation)

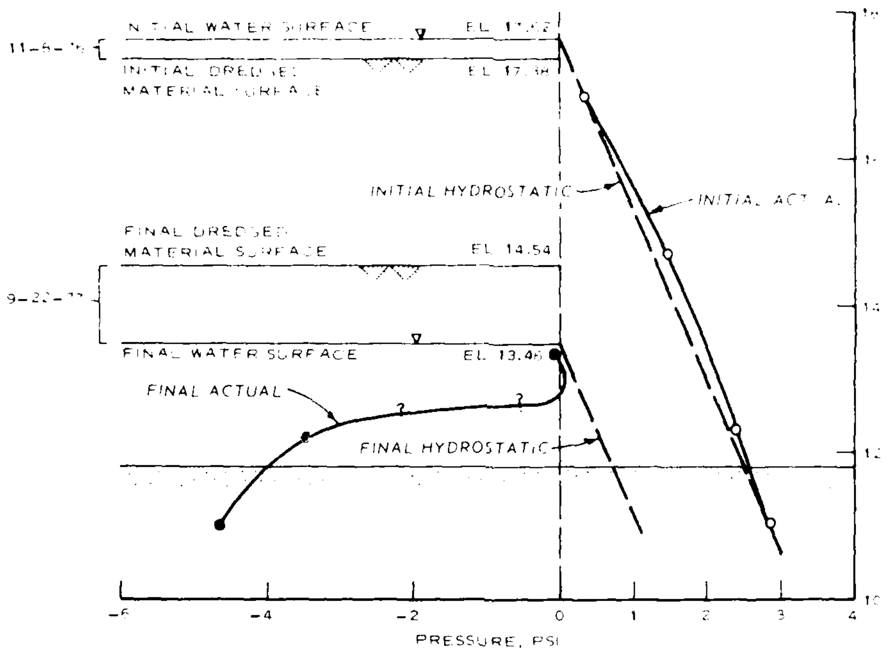


Figure 66. Initial and final pore pressure distribution, test section 3 (partial vacuum), lift 1

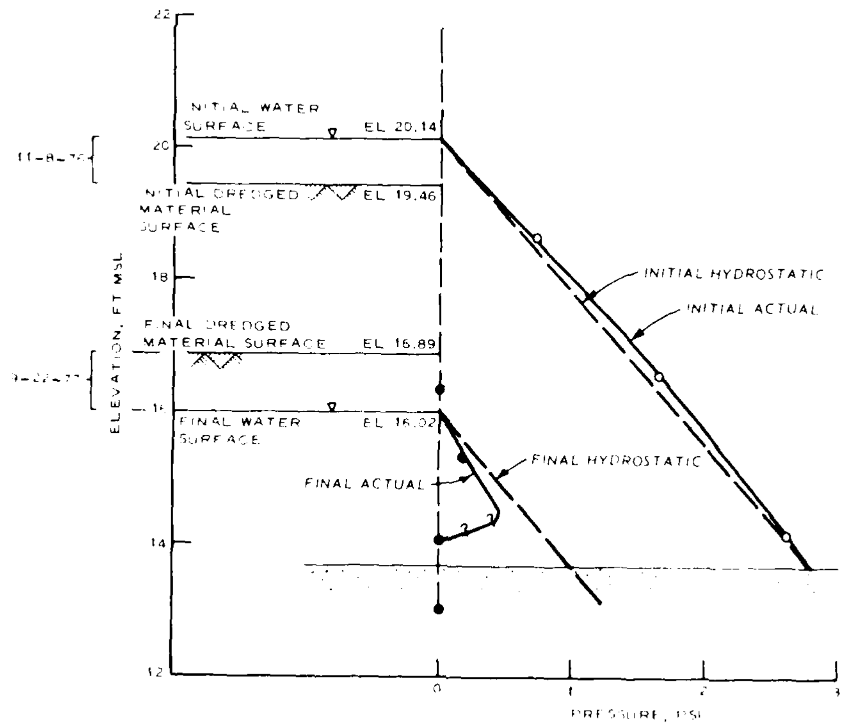


Figure 67. Initial and final pore pressure distribution, test section 4 (gravity), lift 1

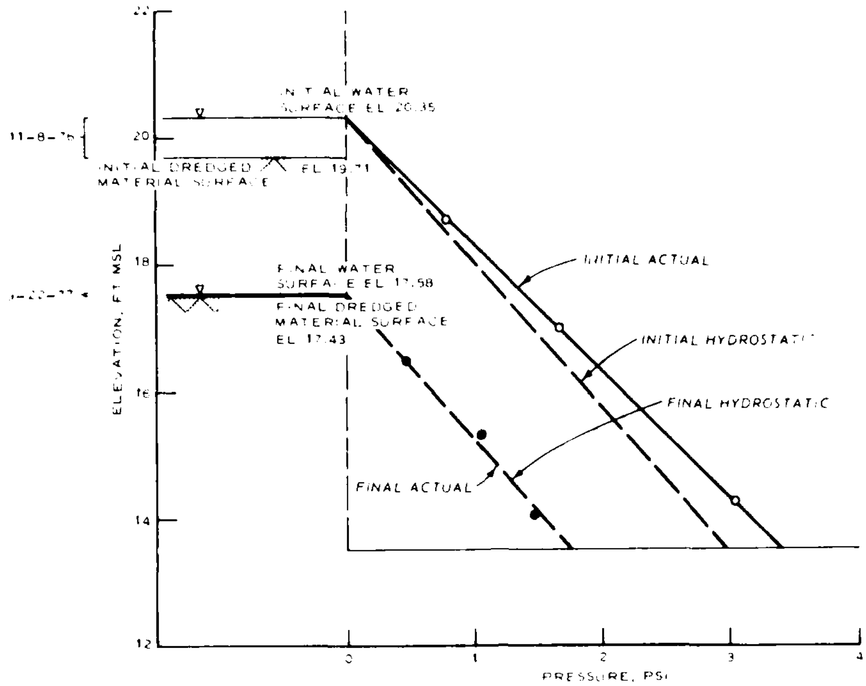


Figure 68. Initial and final pore pressure distribution, test section 5 (control), lift 1

pressure distribution corresponding to the water surface elevation for the initial and final conditions. This is to provide a reference or base to which the actual pore pressure readings can be compared. For some test sections, interpolation was necessary between piezometers where rather drastic changes occurred between instruments. However, since the instuments were only about 2 ft apart in most cases, this interpolation does not detract from the data too much.

121. The final pore pressure distribution curves do not reflect surface drying due to the failure of the piezometers capable of reading negative pressure and because of cracking near the surface around the porous stone [illegible]meter, which exposed them to free water in the cracks, resulting in th[illegible] reading hydrostatic pressure when in actuality the pore pressure in the "blocks" between the cracks was most certainly substantially less than hydrostatic. Also, most test sections did not have an instrument at the exact location necessary to reflect the effects

of dessication. However, readings of all piezometers below this level indicate valid results.

122. The plots contained in Figure 64-68 show that initially pore pressures in excess of hydrostatic pressure existed in all five test sections. However, final pore pressures are shown to be less than hydrostatic for all treated sections but still slightly in excess of hydrostatic in the middle of the untreated (control) section, thus indicating the greater dissipation of pore pressure in the treated sections. Among the treated sections themselves, test section 3 showed the most pore pressure reduction followed by (in order) test sections 2, 4, and 1. The difference between test sections 2, 4, and 1 is not great but the difference between test sections 3 and 2, and 4 and 1 is substantial. This is in accordance with settlement and water content results presented earlier.

Lift 2

123. Plots showing the pore pressure distribution for each of the test sections receiving a second lift are presented in Figures 69-72. These plots are similar to those previously presented for lift 1 and show similar results. Initially, lift 2 for all test sections had pore pressures in excess of hydrostatic pressure but the final distribution curves indicate that only test sections 4 and 5 had any excess remaining. All pore pressures in excess of hydrostatic had dissipated in test sections 2 and 3, both of these sections having intermediate drainage layers. The actual amount of dissipation was greatest in test section 2, followed by sections 3 and 4, with final pore pressures in test sections 3 and 4 being relatively close to final hydrostatic pressure. Test section 2 showed considerably more dissipation than sections 3 and 4. Test section 5, (untreated section) still had pore pressures in excess of hydrostatic. The effect of no drainage at the bottom of test section 5 is readily evident from these data. These pore pressure data conform to previously presented water content results. Previous data showed test section 4 to perform slightly better than 3 while these data show the opposite, but due to the very small difference between sets of data for test sections 3 and 4, the data are considered to be consistent.

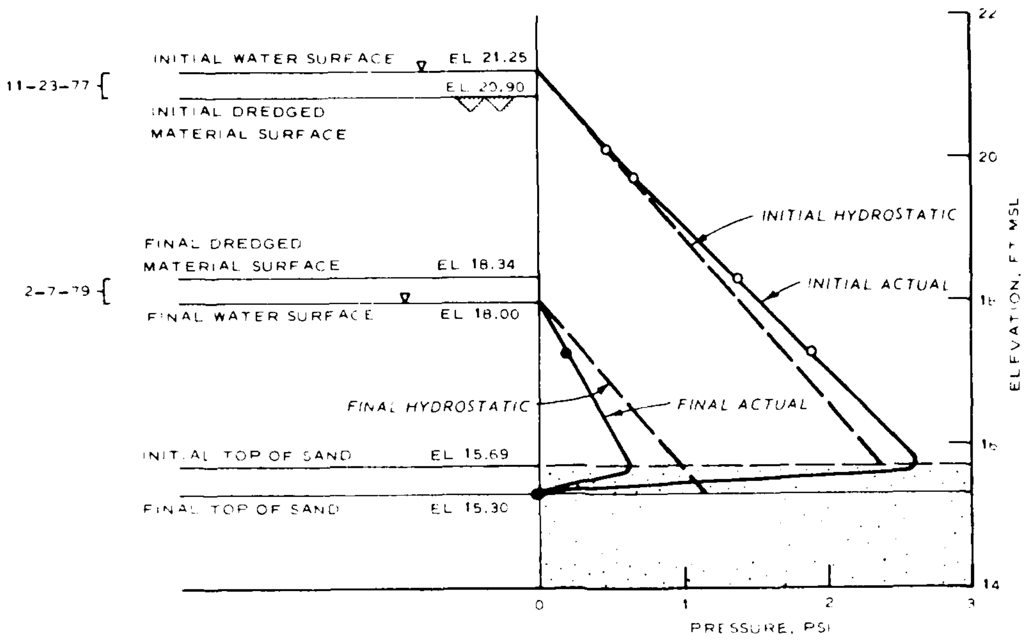


Figure 69. Initial and final pore pressure distribution, test section 2 ("connected" intermediate drainage layer), lift 2

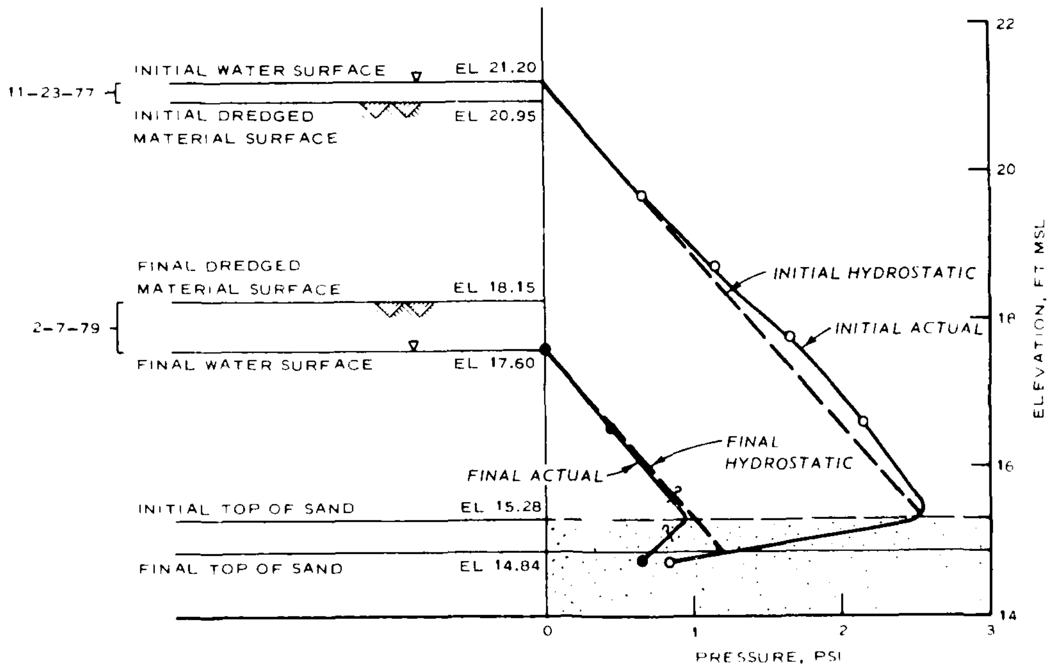


Figure 70. Initial and final pore pressure distribution, test section 3 (intermediate drainage layer), lift 2

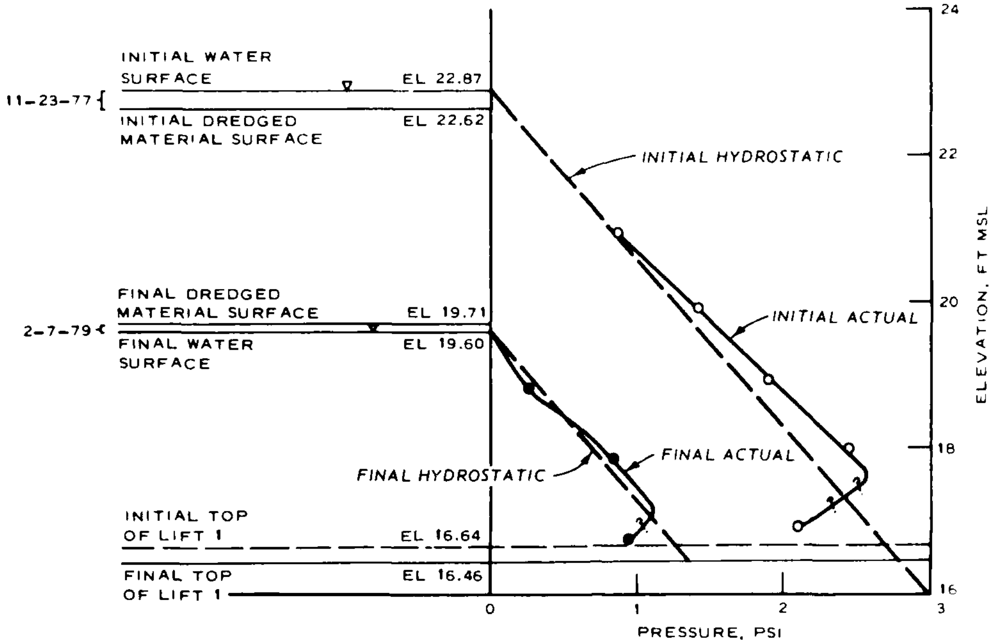


Figure 71. Initial and final pore pressure distribution, test section 4 (gravity), lift 2

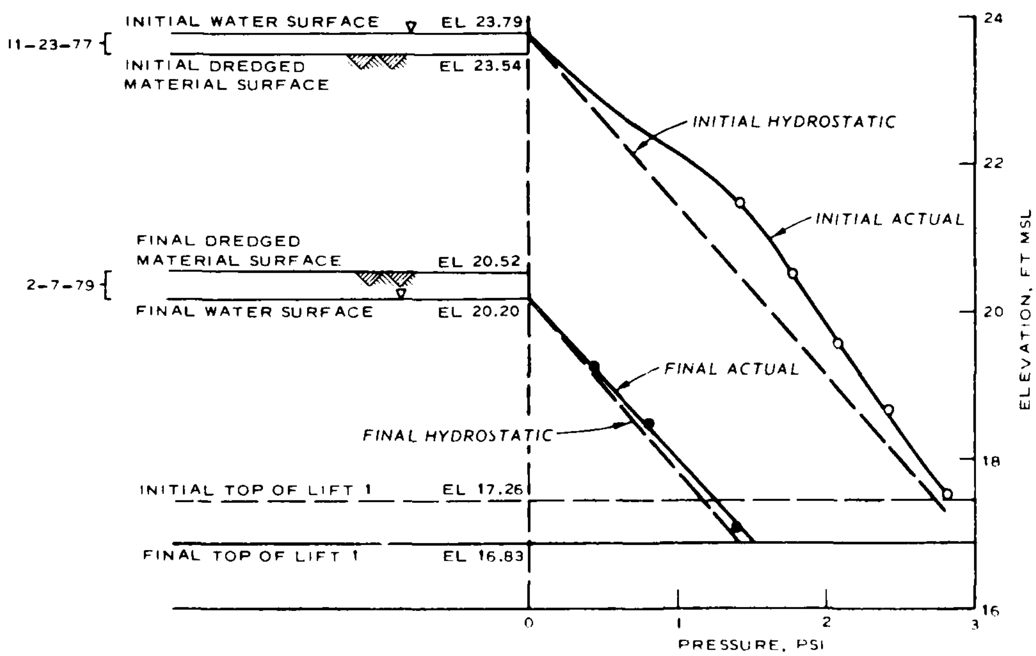


Figure 72. Initial and final pore pressure distribution, test section 5 (control), lift 2

Shear Strength

124. As described earlier in this report, strength determinations were made approximately every 2 months with the vane shear device. The strengths obtained in this manner were unconsolidated, undrained (UU) strengths and should not be considered as the true strength of the material but only as reasonable estimates, and as such are a good basis for judging relative strengths of similar materials. Data for lift 1 end just prior to placement of lift 2 since the sampler could not penetrate through the intermediate drainage layer placed between the two lifts.

Lift 1

125. Results of three intervals of testing (initial, intermediate, and final) in lift 1 are shown plotted with elevation in Figures 73-77 for test sections 1-5, respectively. It is apparent from all plots that the dredged material in all test sections gained strength as the experiment progressed. All plots show a negligible initial strength going to a more or less linear strength profile, increasing from top to bottom, to finally, a parabolic shape indicating maximum strengths near the bottom and surface with minimum strengths near the middle of each layer. In most cases, highest strengths were at the bottom of each layer near the underdrainage layer. Although direct correlations were not made with the water content data, the strength gain and moisture loss do parallel each other in that both sets of curves are similar in general shape. The lesser amount of surface drying received in test sections 1 and 2 is evident from comparisons of their final strength profiles with those for test sections 3, 4, and 5.

126. For ease of comparison all final strength profiles are plotted in Figure 78. This plot clearly shows the greater strength gain obtained in test section 3 (partial vacuum in underdrainage layer) over all the others. It also shows that all treated sections had substantially greater strength gains than did the untreated control section, especially in the middle of each layer. Test sections 1, 2, and 4 have essentially the same profiles (neglecting the effects of surface drying) except that test section 2 (vacuum-assisted seepage consolidation)

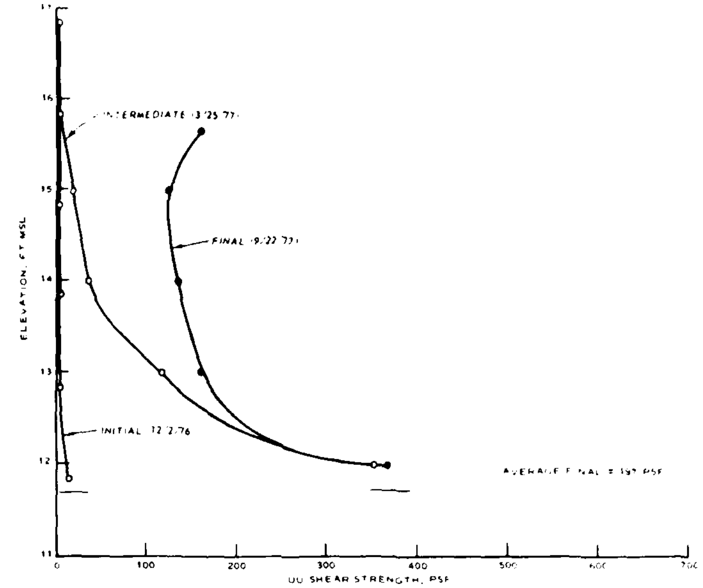


Figure 73. Initial, intermediate, and final shear strengths, lift 1, test section 1 (seepage consolidation)

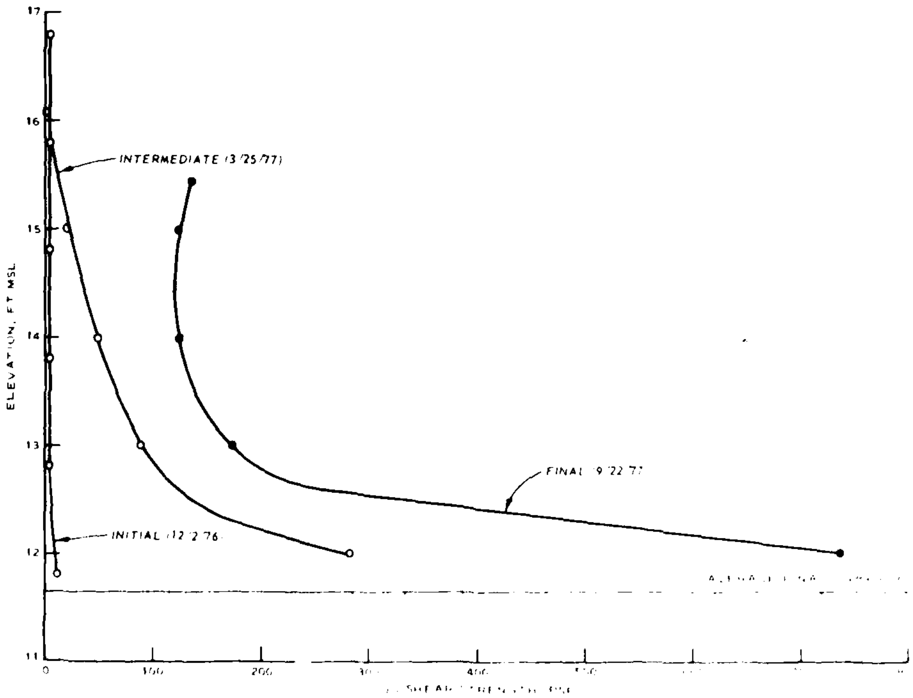


Figure 74. Initial, intermediate, and final shear strengths, lift 1, test section 2 (vacuum assisted seepage consolidation)

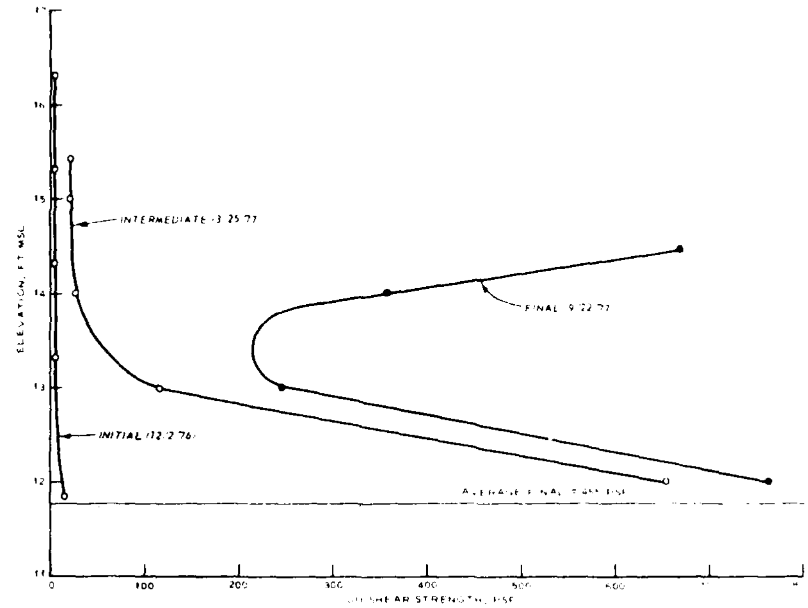


Figure 75. Initial, intermediate, and final shear strengths, lift 1, test section 3 (partial vacuum)

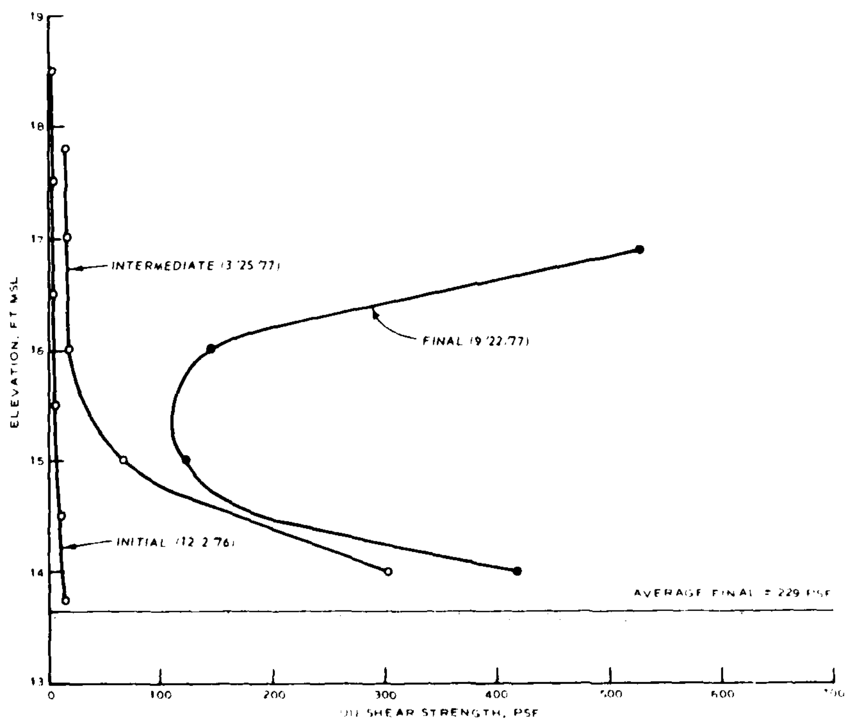


Figure 76. Initial, intermediate, and final shear strength, lift 1, test section 4 (gravity)

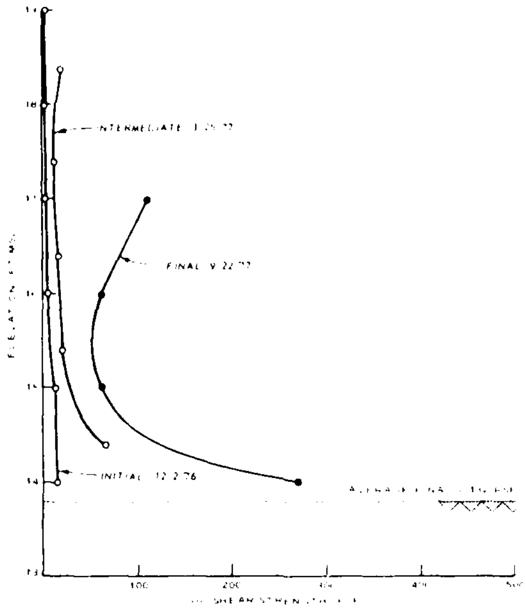


Figure 77. Initial, intermediate, and final shear strengths, lift 1, test section 5 (control)

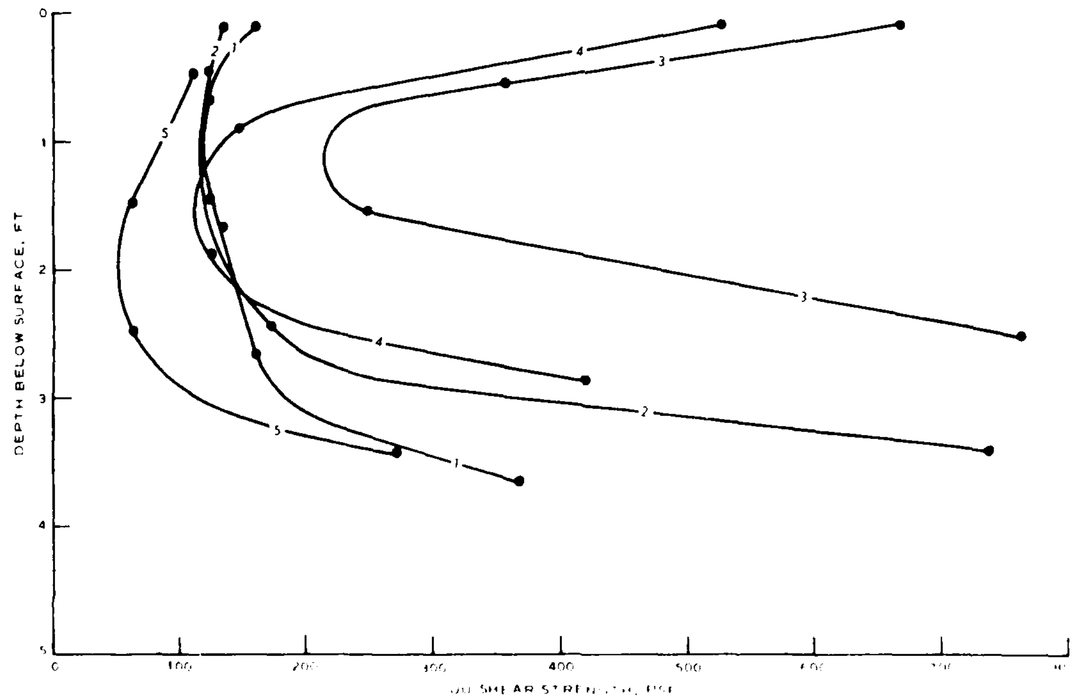


Figure 78. Final shear strengths, lift 1, all test sections

showed a substantially greater increase near the underdrainage layer. As a matter of fact, both test sections having partial vacuum in the underdrainage layer exhibited almost double the strengths in the material near the underdrainage layer than did the other treated sections which had no applied vacuum.

Lift 2

127. Plots of elevation versus shear strength for lift 2 are presented in Figures 79-82. The initial profiles for this lift were established from values obtained from testing nearly 2 months after placement. This, combined with the fact that more initial drainage was available (due to being placed on existing dredged material rather than an impervious bottom), caused an apparent higher initial strength to occur near the bottom than was indicated in the initial strengths of lift 1.

128. The final strength profiles for lift 2 material are more vertical (except for test section 2) than parabolic in shape. This is because of the reduced underdrainage and lack of surface drying (final sampling was in February and followed prolonged rains). The shape of the final strength profile for test section 2, coupled with the low strengths, is somewhat of an enigma when viewed in light of its settlement, pore pressure, and water content data. It is possible that the results obtained at el 16.35 were inordinately low for one reason or another, which could change the shape of the curve as well as increase the final average considerably.

129. All final strength profiles for lift 2 are plotted in Figure 83. This plot shows that all treated sections exhibited greater final strengths than did the untreated control section. It also shows, as has all the other data, that sections 3 and 4 performed similarly but, unlike the other data, test section 2 performed the poorest of the treated sections.

Summary of shear strength

130. A summary of all final average strengths is presented in Table 10. The average strength is an average of all strengths making up the final profile except the surface strength. The surface strengths were omitted because the purpose of the summary is to allow a comparison

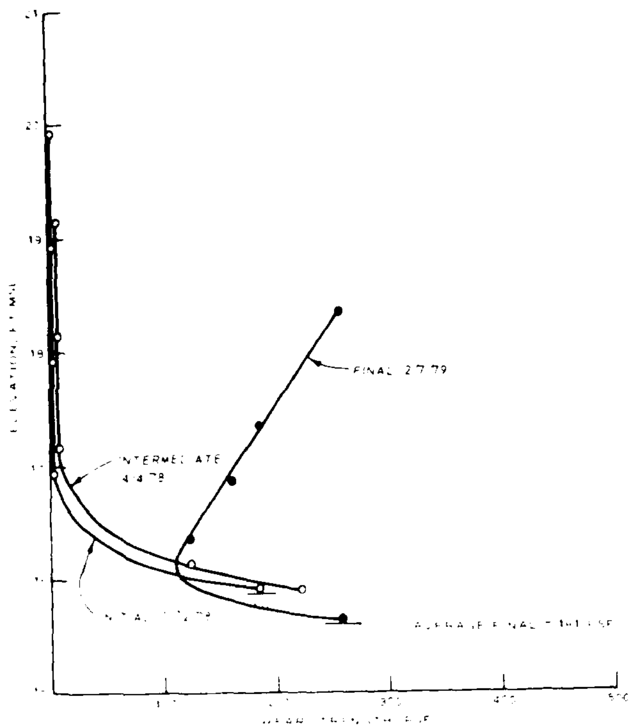


Figure 79. Initial, intermediate, and final shear strength lift 2, test section 2 ("connected" intermediate drainage layer)

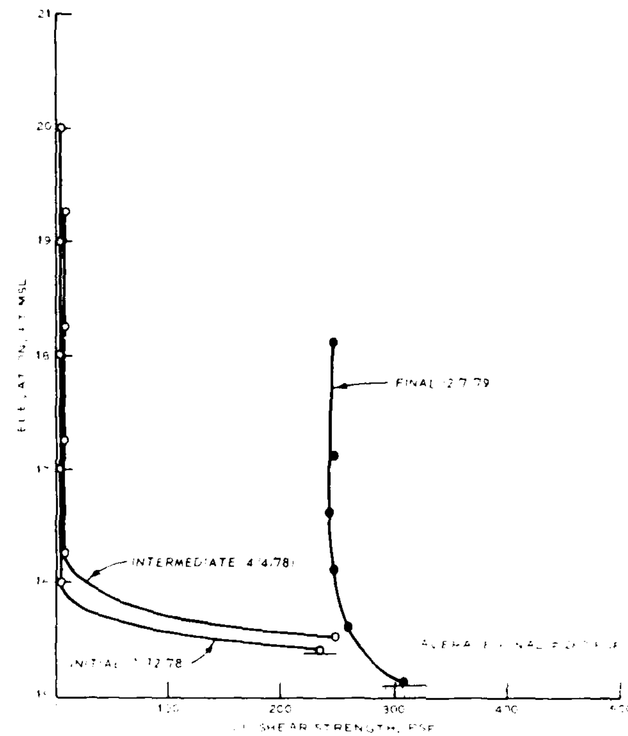


Figure 80. Initial, intermediate, and final shear strength lift 2, test section 3 (intermediate drainage layer)

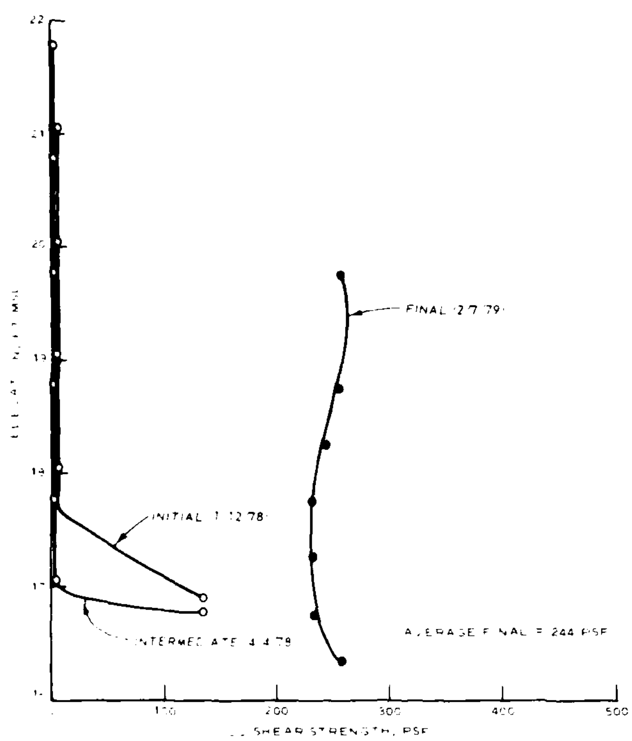


Figure 81. Initial, intermediate, and final shear strengths, lift 2, test section 4 (gravity)

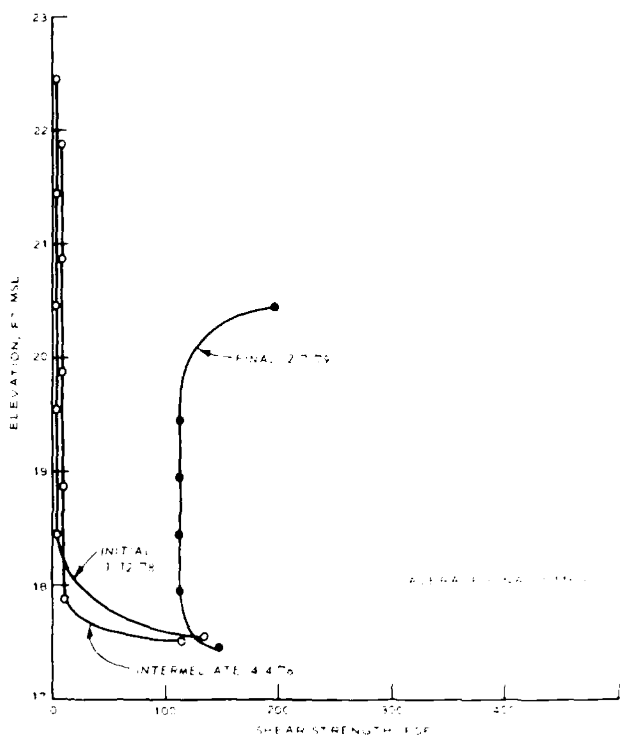


Figure 82. Initial, intermediate, and final shear strengths, lift 2, test section 5 (control)

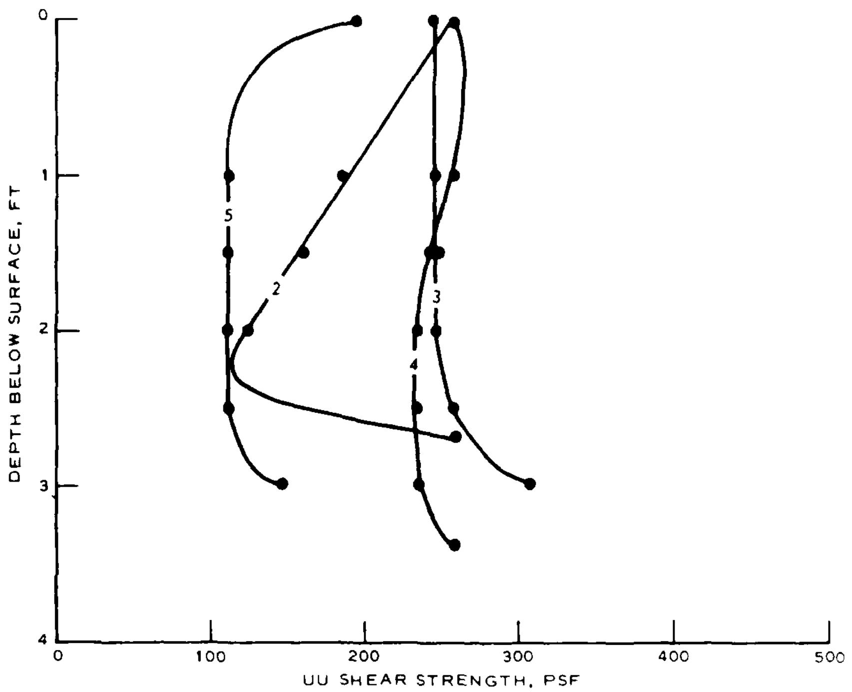


Figure 83. Final shear strengths, lift 2, all test sections

of strengths in order to better evaluate the effectiveness of the underdrainage techniques themselves. Since the surface strengths are a result of surface drying, which was somewhat variable, it was felt that inclusion of these values in the average values would not permit as true a comparison of the underdrainage techniques as would otherwise be possible.

131. Along with the strengths, a ranking of the various techniques studied according to developed strengths is also presented in Table 10.

Summary

132. A ranking of the techniques evaluated according to their effectiveness based on settlement, water content, pore pressure, and vane shear data is presented in Table 11. According to this ranking, the partial vacuum in the underdrainage layer technique was the most effective, followed by gravity drainage, and seepage consolidation.

133. The data actually showed that all techniques evaluated were effective in accelerating densification of the dredged material but, in fact, only one method, partial vacuum in the underdrainage layer, really stood out as clearly superior to the others. This was particularly true for lift 1 and is substantiated by visual observations prior to pumping lift 2, which revealed extensive cracking throughout the entire depth of material. Overall, there was very little difference in the other methods evaluated with the first lift.

134. For lift 2 both test sections 2 and 3 can be considered as representative of a partial vacuum in the underdrainage layer because the only difference in the two was the method of propagating the vacuum from the lower drainage layer into the intermediate drainage layer. The data from lift 2 indicated that both these test sections were more effective than the others, although not by as wide a margin as occurred with lift 1. This is probably because the applied vacuum was much smaller for lift 2 than for lift 1.

PART VIII: SUMMARY AND CONCLUSIONS

135. All underdrainage techniques evaluated were effective to varying degrees in accelerating and/or increasing densification of a 6-ft layer (first lift) of highly plastic dredged material.

136. All techniques evaluated using a second 6-ft-thick lift also resulted in more densification than did the untreated section. However, their effectiveness was not nearly as pronounced in the second lift as in the first.

137. Of the techniques evaluated, the method of applying a partial vacuum in the underdrainage layer was the most effective, while seepage consolidation was the least effective.

138. Benefits realized from application of these underdrainage techniques occurred in the early stages of the experiment. This is due to the very quick consolidation of the dredged material immediately overlying the drainage layer that resulted in the formation of a low permeability layer that essentially controlled the drainage rate of the dredged material above it. This is also the most *likely* reason why the seepage consolidation method was least effective since formation of this low permeability layer would most affect the seepage consolidation method. The effectiveness of all methods could be improved (possibly substantially) by providing better drainage into the underlying drainage layer. This could be accomplished by installation of vertical drains extending through the entire thickness of dredged material into the underdrain.

139. The two configurations attempted with the second lift involving placement of an intermediate drainage layer connected by vertical columns and sand filled cracks to the lower drainage layer were not as effective as anticipated. However, it does appear that the method of using vertical sand connectors does have promise and may be effective if the connectors are more closely spaced and greater care is exercised during installation to ensure they are continuous.

140. If an underdrain system with collector pipe is planned for a project, the addition of a vacuum to the system will most likely be beneficial and cost effective. The results of this experiment [illegible]

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method does work if it is installed in a closed system. Also, the only additional first cost would be that for the pumps and valving, which is considered minimal when compared to the overall cost of the underdrain itself. The only drawback is the fact that energy is required to operate the pumps, and the system would require additional maintenance.

141. The design of an underdrainage system similar to the ones evaluated by this experiment will definitely be site specific, but calculations for designing the system (i.e., collector pipe size and spacing and drain thickness) can be made without difficulty (Cedergren 1977, and Department of the Army, Office, Chief of Engineers 1971a). Also, preliminary design and installation procedures for these systems are given by Haliburton (1978). Rough criteria for the prediction rate of dewatering are also given by Haliburton (1978). More precise procedures are given by Hayden (1978).

142. A considerable amount of field data concerning the behavior of the dredged material under the various applied loading and boundary conditions exists as a result of this experiment. These data have been examined herein only from the standpoint of evaluating and comparing the effectiveness of the techniques studied. However, these data could provide an excellent basis for an engineering analysis of the behavior of the material with the end result being the development of an analytical model, which would be extremely useful in the planning and design of these systems. It is therefore recommended that this work be accomplished.

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Table 1

Rating of Densification Techniques Studied for Field Evaluation

Technique	Rating (1 = Best, 4 = Worst)				
	Increase in Effective Stress over No Treatment*	Relative Cost*	Prototype Applicability	Construction Practicalities	Overall Rating
Underdrainage (constructed sand blanket and collector system)	4	2	1	2	2.25
Underdrainage (collector system installed in pervious foundation)	4	1	1	1	1.75
Seepage consolidation (10-ft-deep surface ponding with underdrainage)	3	2	1	2	2.00
Seepage consolidation (10-ft-deep surface ponding with underdrainage and vacuum)	1	3	2	2	2.00
Temporary earth surcharge (5-ft-high earth fill with underdrainage)	3	3	3	3	3.00
Temporary earth surcharge (10-ft-high earth fill with underdrainage)	2	4	4	4	3.50
Temporary water surcharge (8-ft-deep water with membrane on top of dredged material and underdrainage below)	3	4	2	2	2.75
Temporary water surcharge (16-ft-deep water with membrane on top of dredged material and underdrainage below)	2	4	3	4	3.25
Partial vacuum in underdrainage layer	1	3	1	2	1.75
Partial vacuum in surface drainage layer	2	4	1	2	2.25

* See Johnson, et al. (1977) for actual values used and assumptions made in obtaining these values.

Table 2

Advantages and Disadvantages of Densification Techniques Selected for Evaluation

Technique	Advantages	Disadvantages
Underdrainage	1. Relatively low cost 2. Sand from required dredging or naturally occuring pervious foundation materials may be utilized 3. Can be used in conjuction with other densification techniques 4. Filters the effluent from dredged material 5. Can be used in conjunction with surface drying and vegetative growth	1. Possible construction problems if placement of pervious material is required 2. Occupies storage space in disposal areas 3. Must have collector pipes in order to be effective
Seepage consolidation	1. Relatively low cost 2. Almost doubles effective stress in dredged material as compared with underdrainage alone 3. Eliminates possible odor problems	1. Requires underdrainage layer 2. Requires higher dikes to contain water 3. Prohibits surface drying and vegetative growth 4. Dike erosion from wave action could be a problem
Partial vacuum in underdrainage layer	1. Cost of adding vacuum pumps to underdrain system is low 2. Results in very high effective stress in dredged material 3. Can be used in conjunction with other densification procedures 4. Does not prohibit surface drying or vegetative growth	1. Maintenance required during operation 2. Energy required to operate 3. Possible problems from leakage which could lessen magnitude of desired vacuum

Table 3

Comparison of Effective Stresses

Densification Technique	Effective Stress, psf	Effective Stress Increase, psf
None	140	0
Underdrainage	455	315
Seepage consolidation	770	630
Partial vacuum in underdrainage layer	1175	1035
Vacuum assisted seepage consolidation	1490	1350

Table 4

Summary of Filtration Test Results

Test No.	Drainage Material Tested*	Remarks
1	Filter cloth (openings equivalent to No. 70 sieve size) over pea gravel	Very little penetration of dredged material into pea gravel. No evidence of dredged material in discharge water
2	Filter cloth (openings equivalent to No. 30 sieve size) over pea gravel	Complete penetration of dredged material into pea gravel. Considerable amount of dredged material in discharge
3	Standard concrete sand	Very little penetration of dredged material into sand. Discharge water clear
4	Coarse uniform sand	Considerable penetration of dredged material into sand. Some dredged material visible in discharge water
5	Fine uniform sand (natural material from sand mound in Upper Polecat Bay Disposal Area)	Very little penetration of dredged material into sand; however a small amount of dredged material visiable in discharage water
6	Filter cloth (openings equivalent to No. 100 sieve size) over pea gravel	Some penetration of dredged material into pea gravel, but discharge water clear

* Drainage rates were essentially identical for all tests.

Table 5

Summary of Collector Pipe Data

Test Section No.	Type Drainage	Type Pipe	Diameter of Slotted Pipe, in.	Slot Width, in.	No. Slotted Pipes	Slotted Pipe Spacing, ft
1	Gravity	Schedule 40 PVC	6	0.05	1	30
2	Vacuum	Schedule 40 PVC	1.5	0.01	3	8.5
3	Vacuum	Schedule 40 PVC	1.5	0.01	3	7.5
4	Gravity	Schedule 40 PVC	6	0.05	1	30*

* Since one pipe was utilized in the center of a 30-ft-wide area, this is equivalent to a 30-ft spacing in a larger area.

Table 6

Instrumentation Summary

Parameter to be Measured	Instrument
Positive pore water pressure in dredged material	Porous stone (Casagrande type) piezometer, WES transducer piezometer
Negative pore water pressure in dredged material	Tensiometer, WES transducer piezometer
Positive pore water pressure in underdrainage layer	Porous stone (Casagrande type) piezometer
Vacuum in underdrainage layer	Vacuum piezometer
Settlement of underdrainage layer and foundation	Settlement plate
Discharge from underdrainage layer in test sections 1 and 4 (gravity)	Hourmeter on sump pump
Discharge from underdrainage layer in test sections 2 and 3 (vacuum)	Water meter

Table 7

Final Surface Elevations of Underdrainage Layers

Test Section	Design Elevation msl, ft	Final Elevation,* msl, ft
1	12.0	11.85
2	12.0	11.72
3	12.0	11.80
4	14.0	13.74
5	14.0**	13.53

* Prior to filling with dredged material.
** Bottom elevation (test section 5 had no underdrainage layer).

Table 8

Settlement Summary, Lift 1

Test Section	Original Layer Thickness ft	Current Layer Thickness ft	Net Cumulative Settlement ft	Percent Settlement (or Strain)	Percent Increase Over Test Section 5 (untreated)
		After 321 Days			
1	6.26	3.94	2.32	37.1	4.2
2	6.35	3.78	2.57	40.5	13.8
3	5.48	2.78	2.70	49.3	38.5
4	5.72	3.25	2.47	43.2	21.3
5	6.18	3.98	2.20	35.6	0.0
		After 819 Days			
1	6.26	3.30	2.96	47.3	15.1
2	6.35	3.14	3.21	50.6	23.1
3	5.48	2.62	2.86	52.2	27.0
4	5.72	2.93	2.79	48.8	18.7
5	6.18	3.64	2.54	41.1	0.0

* Computed from $\frac{S_n - S_5}{S_5} \times 100$

where

S_n = percent settlement of test section n(1, 2, 3, or 4)

S_5 = percent settlement of test section 5.

Table 9

Settlement Summary, Lift 2

Test Section	Original Layer Thickness ft	Current Layer Thickness ft	Net Cumulative Settlement ft	Percent Settlement (or Strain)	Percent Increase Over Test Section 5
		After 443 Days			
2	5.37	2.87	2.50	46.6	9.1
3	5.76	2.97	2.79	48.4	13.3
4	6.12	3.25	2.87	46.9	9.8
5	6.44	3.69	2.75	42.7	0

Table 10

Summary of Shear Strength Data

Test Section	Average Final Strength, psf*	Ranking**
	Lift 1	
1	197	1
2	289	2
3	455	1
4	229	3
5	132	5
	Lift 2	
2	181	3
3	260	1
4	244	2
5	118	4

* Neglecting surface strengths.
** 1 = highest.

Table 11

Ranking of Techniques Evaluated Based

On Summary of Field Data Results

Test Section	Technique Evaluated	Type of Data*: Settlement	Water Content	Pore Pressure	Vane Shear
		Lift 1			
1	Seepage consolidation	4	4	4	4
2	Vacuum assisted seepage consolidation	2	3	2	2
3	Partial vacuum in underdrainage layer	1	1	1	1
4	Gravity drainage	3	2	3	3
		Lift 2			
2	Partial vacuum in underdrainage layer, vertical sand column connectors	2	1	1	3
3	Partial vacuum in underdrainage layer, sand-filled cracks	1	3	2	1
4	Gravity drainage, no intermediate drainage layer	2	2	3	2

* 1 = best; 4 = worst.

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Hammer, David F.
Evaluation of underdrainage techniques for the densification of fine-grained dredged material : final report / by David F. Hammer (Geotechnical Laboratory, U.S. Army Engineer Waterways Experiment Station) ; monitored by Environmental Laboratory, U.S. Army Engineer Waterways Experiment Station ; prepared for Office, Chief of Engineers, U.S. Army and U.S. Army Engineer District, Chicago. -- Vicksburg, Miss. : U.S. Army Engineer Waterways Experiment Station ; Springfield, Va. : available from NTIS, 1981.
94, [9] p. : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; EL-81-3)
Cover title.
"March 1981."
"Formerly DMRP Work Unit No. 5A15."
Bibliography: p. 94.

1. Drainage. 2. Dredging. 3. Seepage. 4. Soils - density. 5. Water, underground. I. United States.

Hammer, David F.
Evaluation of underdrainage techniques : ... 1981.
(Card 2)

Army. Corps of Engineers. Office of the Chief of Engineers. II. United States. Army. Corps of Engineers. Chicago District. III. United States. Army Engineer Waterways Experiment Station. Geotechnical Laboratory. IV. United States. Army Engineer Waterways Experiment Station. Environmental Laboratory. V. Title VI. Series: Technical report (United States. Army Engineer Waterways Experiment Station) ; EL-81-3.
TA7.W34 no.EL-81-3

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